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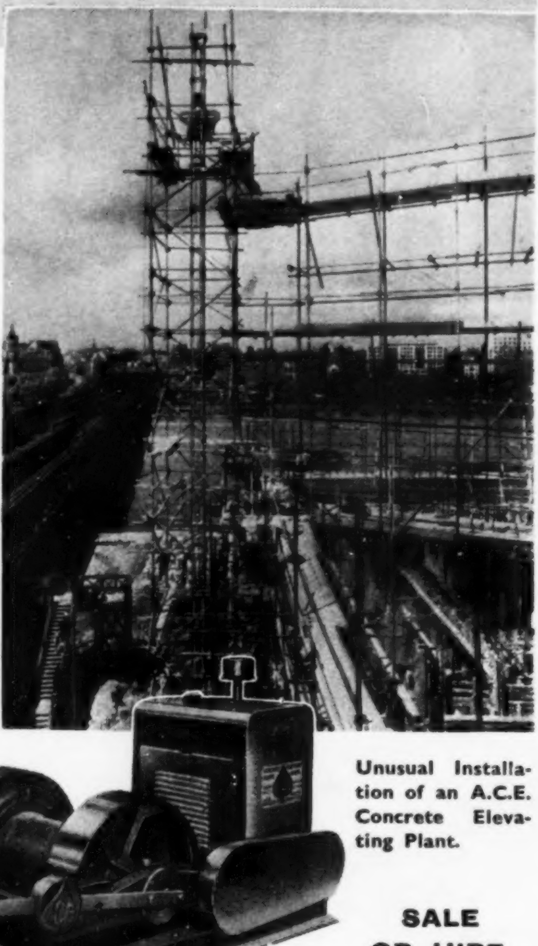

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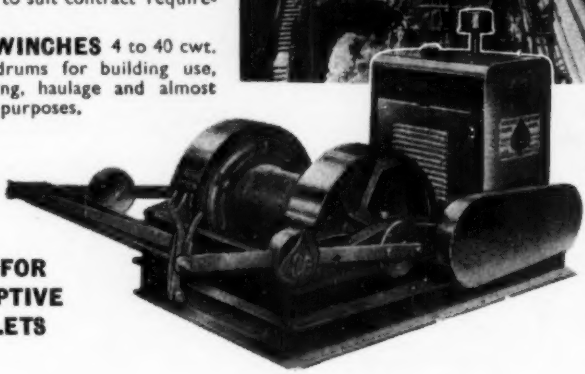

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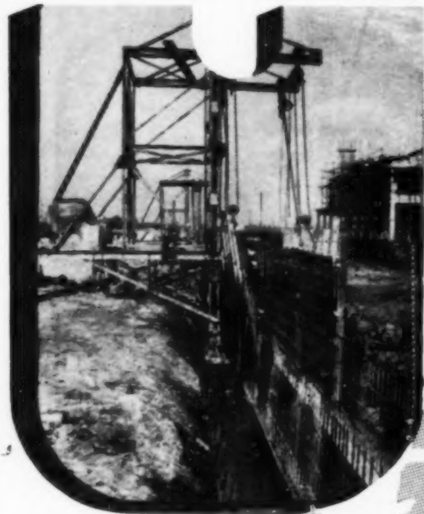

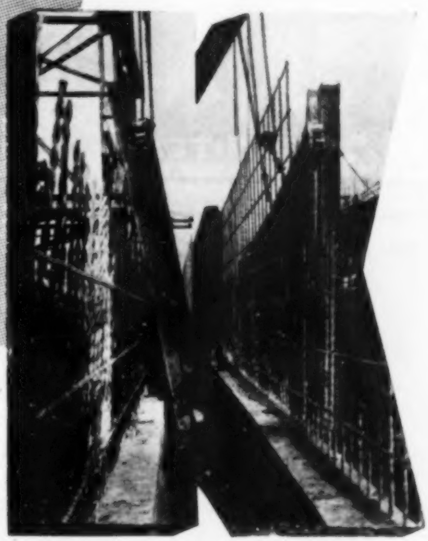

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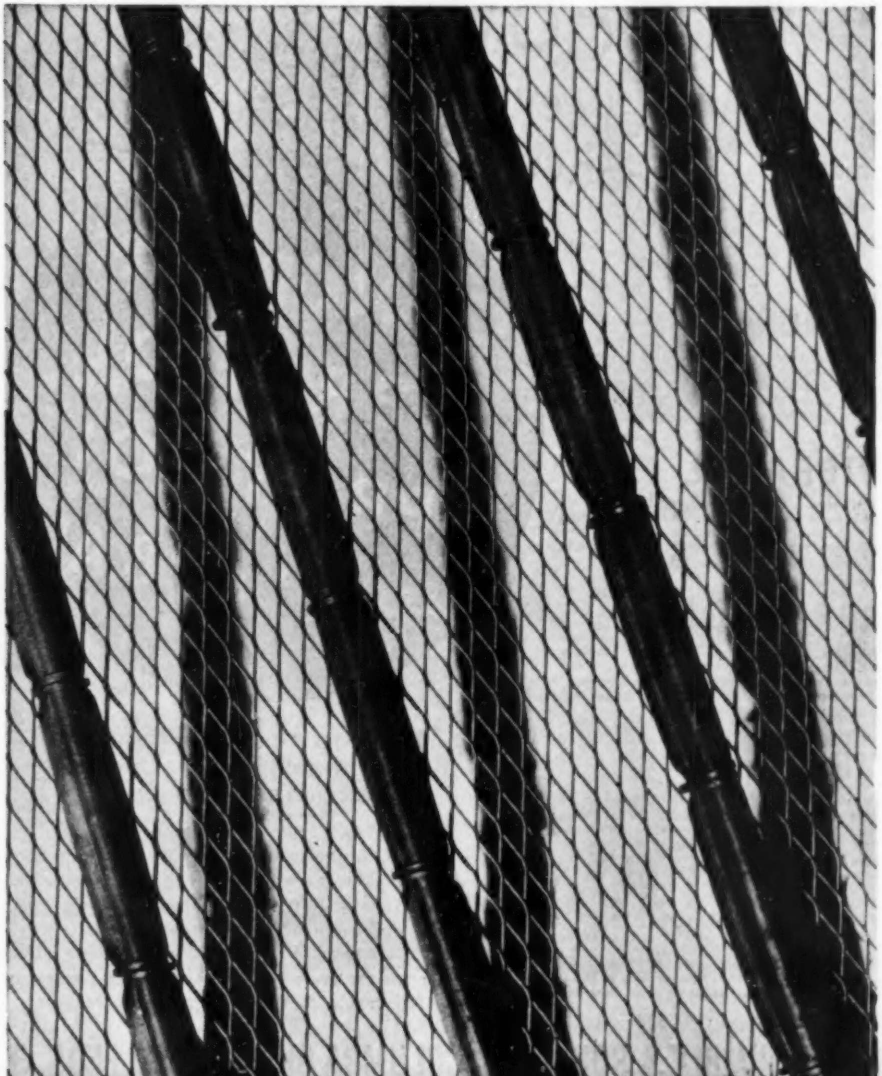

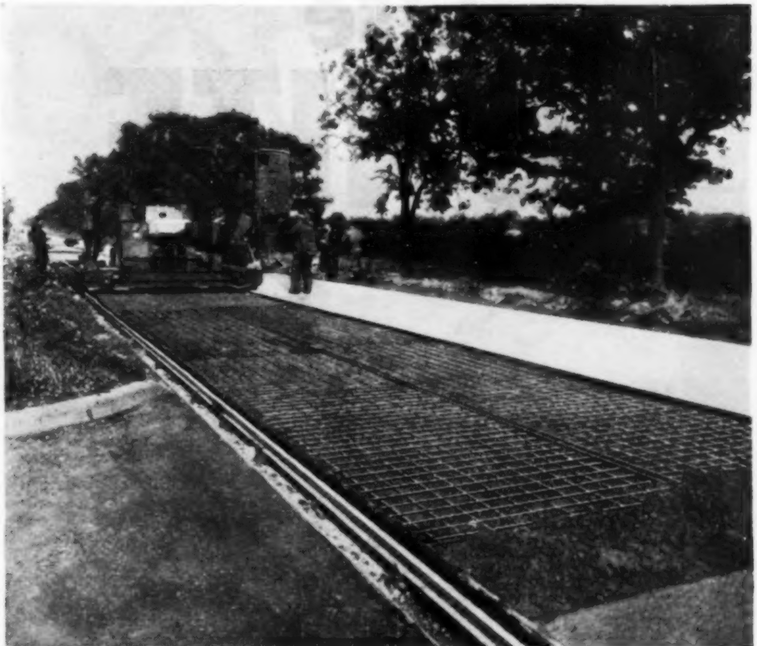

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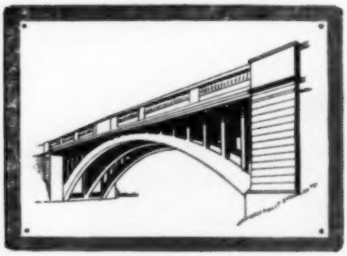

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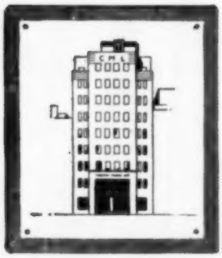
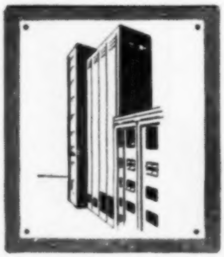
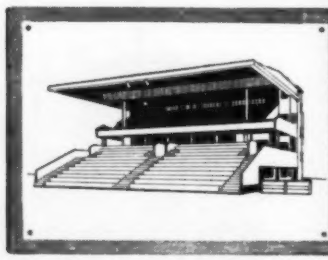

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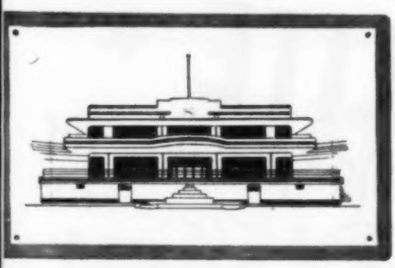

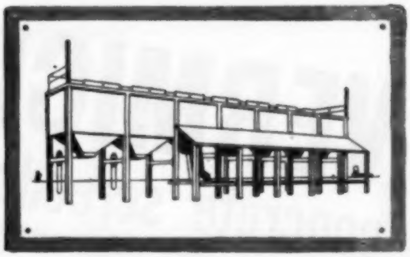

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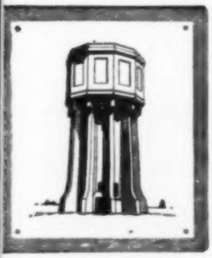

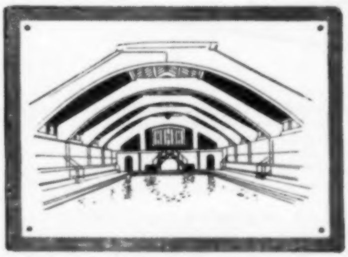

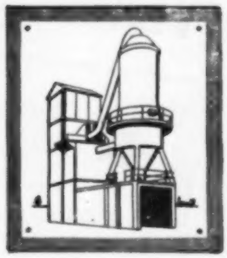

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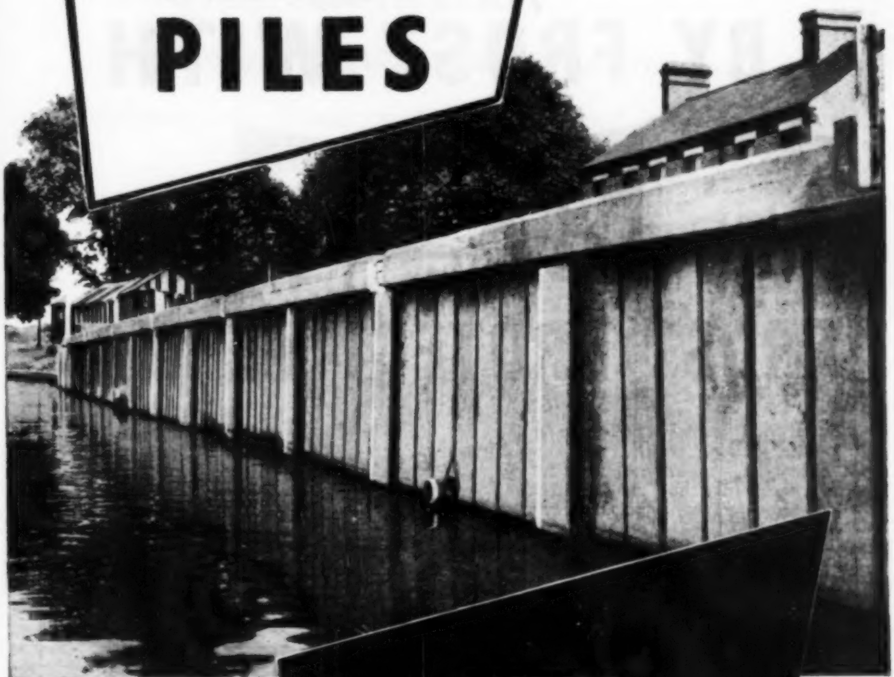

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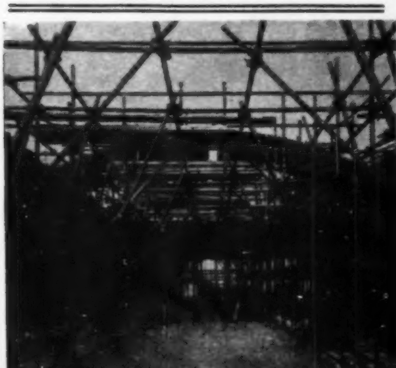

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Volume XLVII. No. 10. LONDON, OCTOBER, 1952

EDITORIAL NOTES

Economy in Mild Steel Reinforcement.

IN our last number we suggested that the publication by the Ministry of Works of the report on "Economy of Building Materials", prepared by the Works Directorates of the three armed Services and of the Ministry of Works, was likely to cause uncertainty in the minds of those responsible for the design of reinforced concrete. This uncertainty is not relieved by the issue by the Ministry of "Steel Economy Bulletin No. 1, The Design of Buildings" (H.M. Stationery Office. Price 3*d*.). The report issued in July refers to the working stresses in mild steel reinforcement in the case of multi-story buildings, and the recommendation, under the heading "Action", is the very definite one to "Work to increased steel stresses in reinforced concrete design, i.e. from 18,000 to 20,000 lb. per square inch". No reason is given why the higher stress is restricted to multi-story buildings. The bulletin was issued in September, and here the recommendation relating to the working stress in reinforcement is: "Stresses higher than those given in C.P. 114 are often justifiable. In the hands of experienced designers a tensile stress of 20,000 lb. per square inch is reasonable for mild steel bars." Here the higher stress is not restricted to multi-story buildings, and the sentence is in the form of a suggestion rather than an instruction.

It is, however, known that the intention of the Ministry is that a working stress of 20,000 lb. per square inch should in general replace the stress of 18,000 lb. recommended in British Standard Code of Practice No. 114. The report definitely states that its findings are expected to be everywhere applied. According to the bulletin, "In future the Ministry of Works will examine licence applications for new buildings in the light of the recommendations of this and any later relevant bulletin, and will normally require a certificate by the architect or the engineer that the recommendations have been fully taken into account." This suggests that there is little likelihood of a licence being granted for reinforced concrete work unless the design allows for a working stress of 20,000 lb. per square inch in the reinforcement, for we may assume that abnormal actions on the part of the Ministry would be infrequent. Also, the hope is expressed that local authorities will have full regard to the recommendations when approving designs under their by-laws, and the Ministry expects that the London County Council will grant waivers to use the higher stress for structures in its area. It thus appears that the Ministry will make every effort to ensure that the higher stress replaces that of the B.S. Code.

The bulletin states that the recommendations are complementary to the B.S. Code. If this is so, and the recommendations are really intended to complete the codes, then there can be no doubt that in the view of the Ministry all users of B.S. Code No. 114 should alter to 20,000 the figure of 18,000 wherever it is given as the permissible working stress in mild steel reinforcement. Indeed, it may be inferred from the sentences quoted that it is now necessary in effect to apply for a waiver for permission to use a working stress lower than 20,000 lb. per square inch, or at least to prove to the Ministry and local authorities that a lower stress is necessary. This is a reversal of the present position whereby it is necessary to apply for permission to use a lower factor of safety; for it appears that special pleading is now necessary if it is desired to maintain a higher factor of safety. It would, by the way, be interesting to know exactly what the Ministry has in mind when in the bulletin it states that, although the codes "have been drawn up with careful attention to economy, their main object is not to secure savings"—if we are careful not to waste material, how do we use less and still conform to good practice?

Most engineers will agree that except in certain types of structures, for example liquid containers, it is reasonable and safe to adopt a working stress of 20,000 lb. per square inch in mild steel reinforcement. It seems, however, that the Ministry is doing the right thing in a wrong way. The code was prepared by experienced people nominated by the learned societies, professional associations, Government departments, and others who are concerned with reinforced concrete and who are anxious that this class of work should be designed and carried out as well as possible. They are concerned with good practice, and good practice includes the economical use of materials. Waste cannot be good practice. The codes are drawn up in a democratic manner by men chosen for their knowledge and experience, and all interested parties are consulted before the documents are issued for the guidance of the profession. So far as the working stress in steel is concerned, B.S. Code No. 114 has been replaced by a recommendation made by representatives of the armed forces and the Ministry of Works, and the Ministry proposes to exercise its powers under the regulations controlling the issue of building licences to enforce the adoption of a higher stress for general reinforced concrete work. It would have been better, and more in keeping with peace-time procedure, if the Codes of Practice Committee had been asked to consider the revision of B.S. Code No. 114. Since this document was issued in 1948 there has been a growing belief that a higher working stress can safely be adopted, and there is little doubt that when the Code is revised it will recommend the same stress that is now asked for by the Ministry. We should then have a recommendation made with the backing of the profession and which would have been adopted by most local authorities without the persuasion which the Ministry now thinks fit to bring to bear. Codes of practice take a long time to prepare, but if the deliberations of the committee were concerned only with the working stress in steel in the existing code there is no doubt that a decision could have been quickly reached, and it is likely that the recommendation would have had a result similar to that which the Ministry now requires. There was no need to array the nation's fighting forces and the Ministry in order to convince engineers that a higher stress in steel could be safely adopted. Nothing is gained by using force to make people do what they would do of their own free will.

Calculation of Shell Roofs without Stiffening Beams.

By ANDRÉ PADUART (BRUSSELS).

AT the Symposium on Concrete Shell Roof Construction held in London in July last a description * was given of a large shed built by the Belgian firm S.E.T.R.A. at Antwerp. The shed is 1525 ft. long and 199 ft. wide and is covered by thin reinforced concrete vaults of 50 ft. span. The thickness of the vaults increases from 3 in. at the crown to 5 in. at the springings. A notable feature is the absence of tie-rods and stiffening beams. The thrust of the vaults is resisted at each end of the building by reinforced concrete counterforts and there are no expansion joints in the structure.

The calculation of the stresses in these vaults according to the general theories of elasticity is extremely difficult as the bending moments normal to

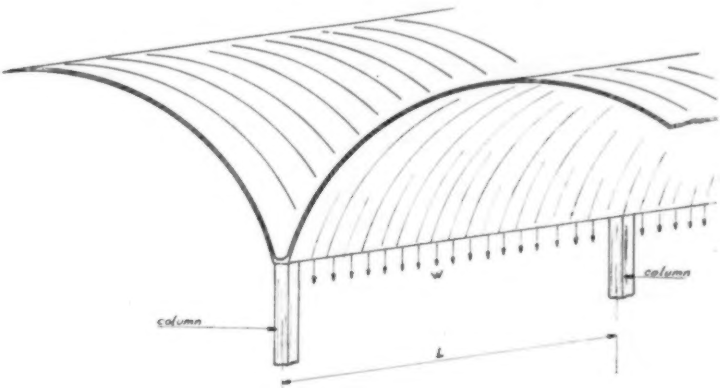


Fig. 1.

the surface of the shell are not negligible. It would have been possible to have established differential equations giving the stresses at each point and to integrate them, taking the conditions at the edge into account, but even by using Poisson's ratio equal to zero it would have been necessary to introduce very important approximations. A more practical method was preferred and the reinforcement was determined in a much easier way. This method comprises two successive stages.

In the first stage it is assumed that each valley is supported along its whole length. The problem is then related to a two-dimensional arch instead of a three-dimensional vault, and the corresponding stresses can be easily calculated at any point by the theory of elastic arches. Wind, non-uniformly-distributed snow loads, shrinkage, and temperature variations cause deflections which produce very slight tensile stresses in the concrete. Although these do not exceed the permissible stresses, a mesh reinforcement weighing 0·62 lb. per square foot is placed in the vaults. In the second stage it is assumed that there are no

[* An abstract of this paper is given on p. 311 of this number.]

props except the columns along the whole length of the valleys. In this stage the stresses in a vault have to be calculated, and a three-dimensional problem must therefore be solved. The loads acting on the vaults in this stage are only the forces w on the temporary supports of the preceding stage (*Fig.* 1).

These forces w are resolved into two forces w_1 acting in planes tangential to the vaults (*Fig.* 2). It is therefore necessary to consider two beams B_1 of great depth and of small thickness carrying the loads w_1. The only special problem to be solved is the determination of the effective depth of these beams; this is a function of the span L and the width l of the vaults. It was considered reasonable to assume the effective depth H as the smaller of the values $0{\cdot}3L$ or $0{\cdot}4l$.

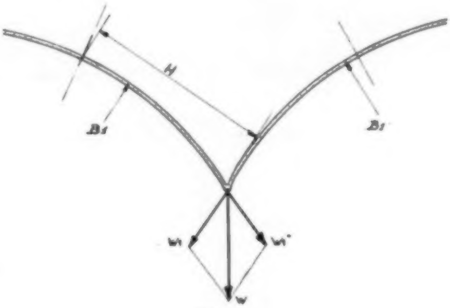


Fig. 2.

If the maximum bending moment M_1 is $\frac{w_1L^2}{8}$, and S denotes the permissible stress in the reinforcement, the necessary area of steel A in each springing is given by $\frac{w_1L^2}{8} \times \frac{1}{0{\cdot}87H} \times \frac{1}{S}$. Assuming that $H = 0{\cdot}3L$,

$$A = \frac{w_1L}{2{\cdot}1S} \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

This formula is very similar to that given in 1935 by Issenmann Pilarski * which can be written

$$A = \frac{w_1L}{2S} \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

Another way of calculating the reinforcement consists in considering the two half-shells joined together in the valley as a vertical V-shaped beam carrying the load w (*Fig.* 3). The maximum bending moment M on this beam is $\frac{wL^2}{8}$, and the total area of steel $2A$ required in the valley is $\frac{wL^2}{8} \times \frac{1}{0{\cdot}9h} \times \frac{1}{S}$. Hence

$$A = \frac{wL^2}{14{\cdot}4hS} \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

* "Calcul des voiles minces en béton armé." Dunod, Paris.

Applying these three formulæ to the vaults at Antwerp we have : $L = 57$ ft., $l = 49$ ft., and $h = 10$ ft.

$$w_1 = \begin{cases} \text{thrust due to dead weight of the vaults,} & 1780 \text{ lb. per ft.} \\ \text{thrust due to snow,} & 280 \text{ lb. per ft.} \\ \text{weight of the gutter,} & 154 \text{ lb. per ft.} \end{cases}$$

Total 2214 lb. per ft.

$w = 2760$ lb. per foot. $S = 17{,}000$ lb. per square inch.

$$\text{Formula (1) gives } A = \frac{2214 \times 57}{2 \cdot 1 \times 17{,}000} = 3 \cdot 52 \text{ sq. in.}$$

$$\text{Formula (2) gives } A = \frac{2214 \times 57}{2 \times 17{,}000} = 3 \cdot 72 \text{ sq. in.}$$

$$\text{Formula (3) gives } A = \frac{2760 \times 57^2}{14 \cdot 4 \times 10 \times 17{,}000} = 3 \cdot 66 \text{ sq. in.}$$

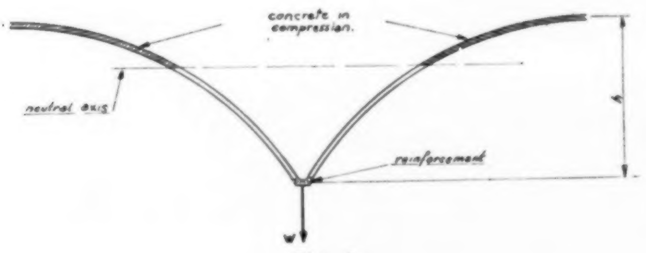


Fig. 3.

The reinforcement used consists of seven ¼-in. bars of the mesh and eleven ⅝-in. bars. These bars are bent upwards at 45 deg. near the columns in a strip 21 ft. wide in order to resist the shearing forces and negative bending moments (*Fig.* 4—this is *Fig.* 3 on page 312 of this number).

Because of the absence of stiffening beams, the resistance to buckling of the vaults was investigated. The critical thrust is given by $T_e = F_e \cos \alpha$ where F_e is the buckling load of an equivalent straight beam calculated by the Euler formula, and α is the slope of the tangent to the vault at the quarter-point of the span.

$$F_e = \frac{\pi^2 EI}{a^2} = \frac{\pi^2 \times 2{,}840{,}000 \times 103}{310^2} = 30{,}400 \text{ lb.};$$

$$\cos \alpha = \cos 24 \text{ deg.} = 0 \cdot 913. \quad \text{Hence } T_e = 27{,}700 \text{ lb.}$$

The maximum thrust at the crown of the vault is 5150 lb. The coefficient of security against buckling is thus 5·38.

The first vaults were built more than four years ago and they have behaved quite well in spite of the heavy loads of snow which they have already borne, and the violent storms and the great variations of temperature which they have undergone. Two other sheds have been built more recently according to the same principle. Their length is 850 ft. and their width 160 ft.

A Prestressed Concrete Footbridge.

THE illustrations show one of four prestressed concrete bridges built by the Hampshire County Council to replace worn-out wooden structures. The bridges are 4 ft. wide and have spans up to 43 ft. The beams are made of hollow blocks 4 ft. long by 10 in. square with walls 1 in. thick, and between the blocks are placed solid concrete diaphragms as shown in *Fig.* 2. Two rows of these blocks are joined together by reinforcement bars passing transversely through the solid blocks and comprise one beam, and two of these beams form the deck of a footbridge. The prestressing cable is between the two rows of blocks forming a beam, and is placed between the transverse bars which join together the two rows of blocks and the flanges at the bottom of the beams. Solid blocks are used at the ends of the beams to form anchorages for the cables. The spaces between the rows of blocks are filled with mortar, and the bridges are surfaced with asphalt. The blocks can be carried by one man, and three or four men only are required to erect a bridge.

The bridges were designed by Mr. E. W. Gifford, formerly Bridge Engineer of the County Council, and the contractors were Messrs. J. J. Udalls (Building), Ltd. The blocks were made at the works at Romsey of the Devon Concrete Co., Ltd.

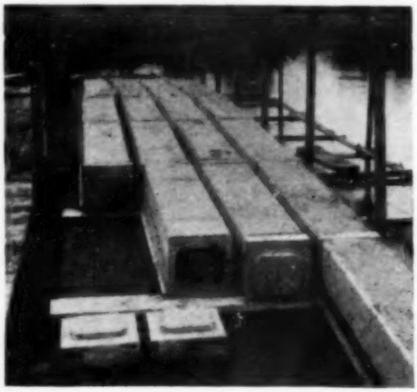
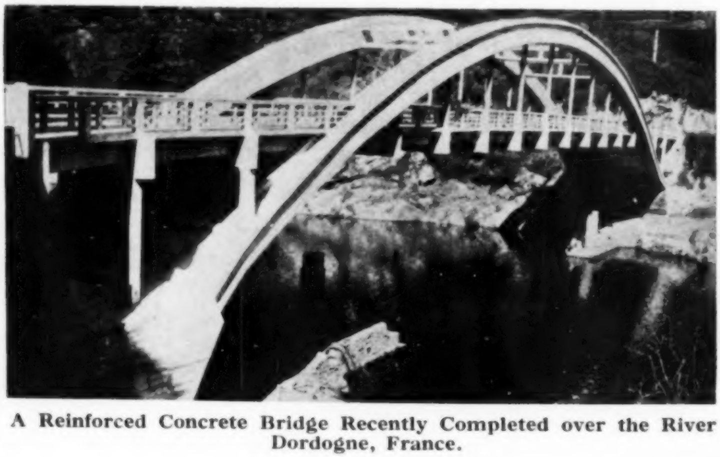


Fig. 2.

Fig. 1.



A Reinforced Concrete Bridge Recently Completed over the River Dordogne, France.

A Culvert with a Prestressed Concrete Deck.

TENSIONED STEEL BARS.

As part of a flood-relief scheme in the valley of the river Lee a new channel is being constructed across Leyton Marshes in north-east London. The channel is about 2000 ft. long and one-half of the length is to be covered. The remainder of the channel is open.

The Channel.

Fig. 1 shows the covered part of the culvert at the outfall and *Fig.* 2 gives cross-sections of the culvert and the open channel. With the exception of the pre-stressed concrete beams forming the deck of the culvert the whole of the work is of plain concrete. The walls of the open channel are 9 ft. 6 in. high and 1 ft. 4½ in. thick at the top, increasing to 3 ft. 3 in. at the base of the wall on the right bank and 4 ft. on the left bank. Each wall has a toe extending 1 ft. 4½ in. by 1 ft. 6 in. deep under the invert slab. The walls are cast in travelling wooden shutters (*Figs.* 3 and 5) 40 ft. long, which is the distance between contraction joints. The shutters are suspended from gantries travelling on rails and are moved by winches, mounted on the framework of the shutter, pulling on wire ropes anchored to the base of the part of the wall previously cast. The shutters are moved along the 40-ft. section of wall twelve hours after casting the concrete.

The joints between each section of wall are the castellated type, the face of one section being painted with bitumen before the next is cast against it. On the channel face of the joint a rebate 3 in. deep by ½ in. wide is formed and filled with 2 in. of an elastic jointing material and 1 in. of bituminous sealing material. The outer face of the wall is rebated in a similar manner and filled with 3 in. of an expansible filler.

The invert slab is 6 in. thick and laid in panels 24 ft. by 20 ft. in the culvert and 27 ft. 6 in. by 20 ft. in the channel, with sealed expansion and contraction joints ⅜ in. wide between the panels. The slab is laid on bituminous felt on a base course of concrete 3 in. thick.

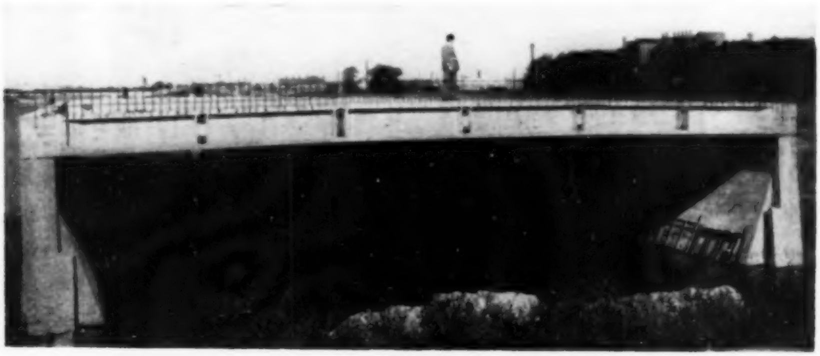
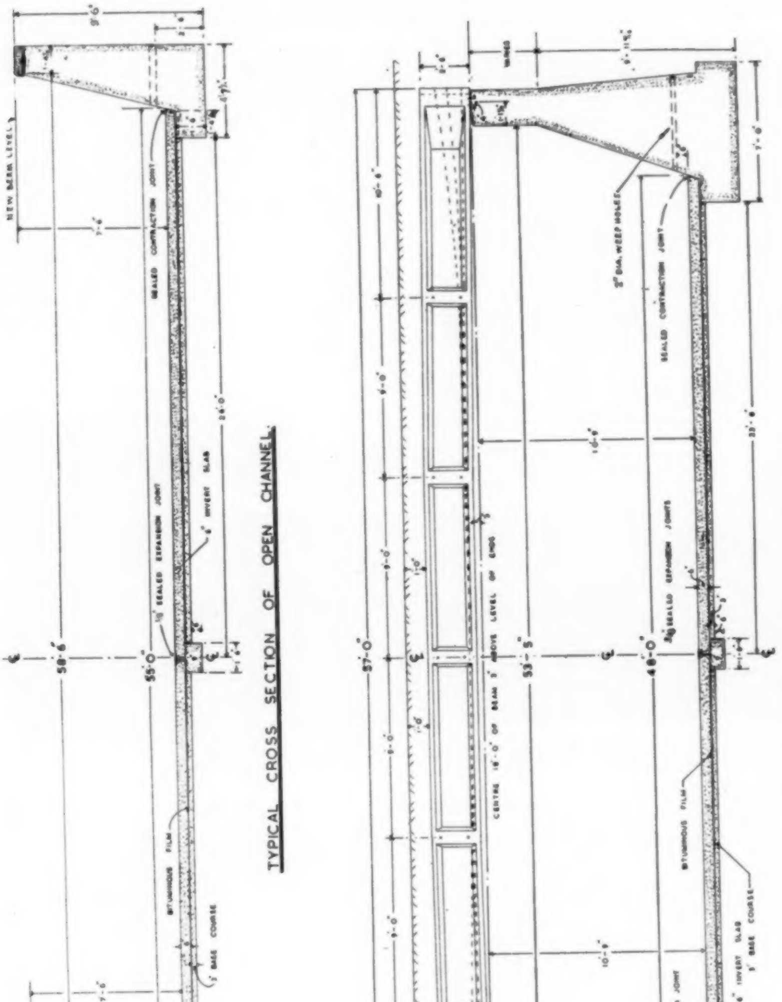


Fig. 1.—Outfall End of Covered Culvert.

The mixture of the concrete for the walls is 1 : 2·5 : 3·4 with a water-cement ratio of 0·55, for the invert the mixture is 1 : 2·8 : 3·8 with a water-cement ratio of 0·5, and for the base slab 1 : 3·1 : 4·2 ; in all cases the maximum size of the aggregate is ¾ in. The concrete is weigh-batched in a plant designed by the contractor (*Fig.* 4) and is mixed in a 21/14 rotary-drum mixer placed over the receiving hopper of a 4-in. concrete pump which distributes the concrete throughout the site. The aggregate is lifted to the hoppers of the batching plant by a crane, which also handles the loose cement containers seen in *Fig.* 4. These containers have a capacity of 3 tons of cement.

The walls of the culvert are similar in design and construction to those of the channel, except that they are about 3 ft.



TYPICAL CROSS SECTION OF OPEN CHANNEL.

TYPICAL CROSS SECTION OF CULVERT.

Fig. 2.—Details of Culvert.

higher in order to allow sufficient free-board. *Fig.* 2 is a cross-section of the culvert and *Fig.* 6 shows the walls before placing the beams.

Prestressed Cover of Culvert.

Alternative designs were prepared for the culvert in reinforced concrete and prestressed concrete but, because the depth of a single-span beam in reinforced concrete was so great that considerable increase in the width of the culvert would have been necessary if the required levels were to be maintained, the final design in reinforced concrete included central piers. Although the estimated cost of the chosen design in prestressed concrete was slightly greater than that for reinforced concrete, the former was chosen in view of the possibility of the piers interfering with the flow of water and the likelihood of debris collecting around them.

The prestressed concrete beams have an overall length of 57 ft., the clear span between the walls being 53 ft. 5 in. The load for which they are designed is the Ministry of Transport Standard Highway Load plus an earth cover of 100 lb. per square foot. The beams are precast and prestressed at a factory at Hoddesdon and are transported by rail to a siding adjacent to the culvert. A temporary railway line has been laid to allow the beams to be taken from this siding to a loading bridge spanning the culvert. The wagons are pushed on to the bridge by bulldozers (*Fig.* 7), and the beams are lifted from the wagons on to a road vehicle by cantilever-type gantries (*Fig.* 8). The lorry, travelling inside the culvert, carries the beams to two steel trestles within a few feet of their final position. The trestles support traversing jacks which raise the beams from the lorry and place them in position on the walls of the culvert (*Figs.* 11 and 13).

After being placed in position the beams are prestressed transversely, in bays twenty beams wide, by ten ⅞-in. diameter "Macalloy" bars which pass through holes previously formed in the webs of the beams (*Fig.* 10). After the bars are tensioned the surplus lengths projecting at the end of each bay are cut off by oxy-acetylene flame and the space between the bars and the concrete is grouted with cement mortar under pressure. The bearing of the beams at one end consists of a pad of 1 : 2 cement mortar, 4 in. wide by ½ in. thick; at the other end it comprises two 18-S.W.G. copper plates placed between mild-steel plates 4 in. wide by ⅜ in. thick. This is shown in *Fig.* 13, which also shows the final adjustment of a beam in position.

The number of beams required is 512. They are of I-section, 30 in. deep by

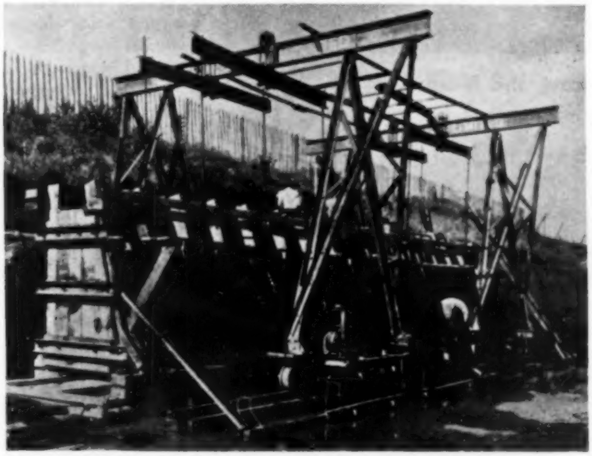


Fig. 3.—Travelling Shutter for Walls.

24 in. wide; the web is 4 in. thick and the flanges vary from $2\frac{1}{2}$ in. at the edges to 5 in. at the junction with the web. Each beam contains three high-tensile Macalloy bars of $1\frac{1}{8}$ in. diameter which are tensioned after the concrete has attained sufficient strength. The only mild steel reinforcement in the beams is a light-gauge mesh of mild steel bars giving an area of 0·14 sq. in. of steel in the top flange of each beam, and $\frac{3}{8}$-in. diameter bars at the ends of the beams to distribute the high stresses immediately in front of the steel plates at the anchorages of the prestressing bars. There are stiffening ribs 8 in. wide at 9-ft. intervals along the beam, through which are formed two $1\frac{1}{2}$-in. diameter holes to receive two $\frac{7}{8}$-in. diameter bars after erection of the beams on the site as already described.

Fig. 4.—Batching Plant.

Making the Beams.

The beams are made in two parts, one 37 ft. $4\frac{1}{2}$ in. long and one 19 ft. $4\frac{1}{2}$ in. long (see *Fig.* 9), to suit the 40-ft. long "shock" tables at the works. The joint is 3 in. wide and is formed at a stiffening rib. The beams are cast in timber moulds and placed on trolleys which are pushed on to a "shock" table, which is lifted through a distance of about $\frac{3}{16}$ in. and dropped on to rigid bearers 200 times a minute by means of cams on rotating shafts under the table. The concrete is placed in shallow layers throughout the length of the mould. About 40 minutes are required to fill a

Fig. 5.—Travelling Shutter for Walls.

mould, and during the whole of this period the shock-table is working. In the case of the larger beams the total weight of the mould and the beam is about 7 tons, and the falling of this heavy load on to the bearers below the table produces concrete of great density and ensures that the concrete is forced into the flanges at the bottom of the mould. The concrete is in the proportions of 1 part of ordinary Portland cement, 1½ parts sand, and 3 parts ¾-in. ballast, and the water-cement ratio is 0·4. The cavities, 1¼ in. diameter, for

Fig. 6.—The Culvert before Placing the Beams.

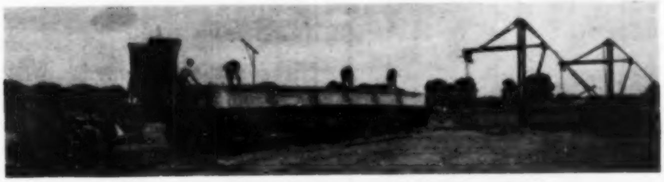


Fig. 7.—Pushing Wagons to Gantry Crane.

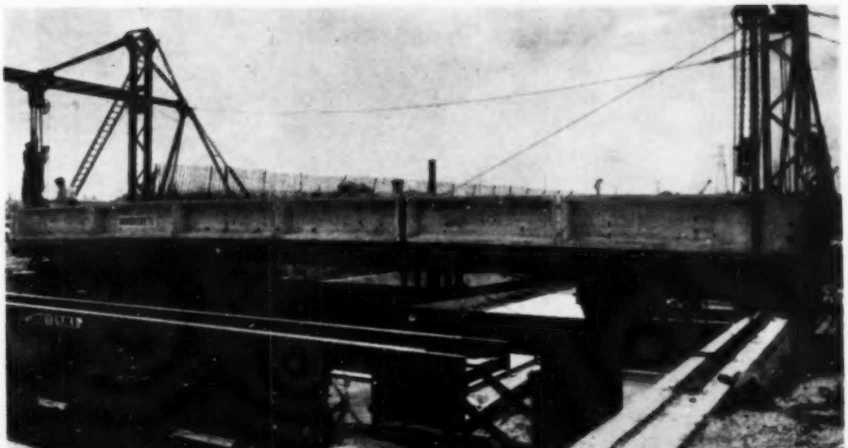


Fig. 8.—Placing Beam on to Road Vehicle.

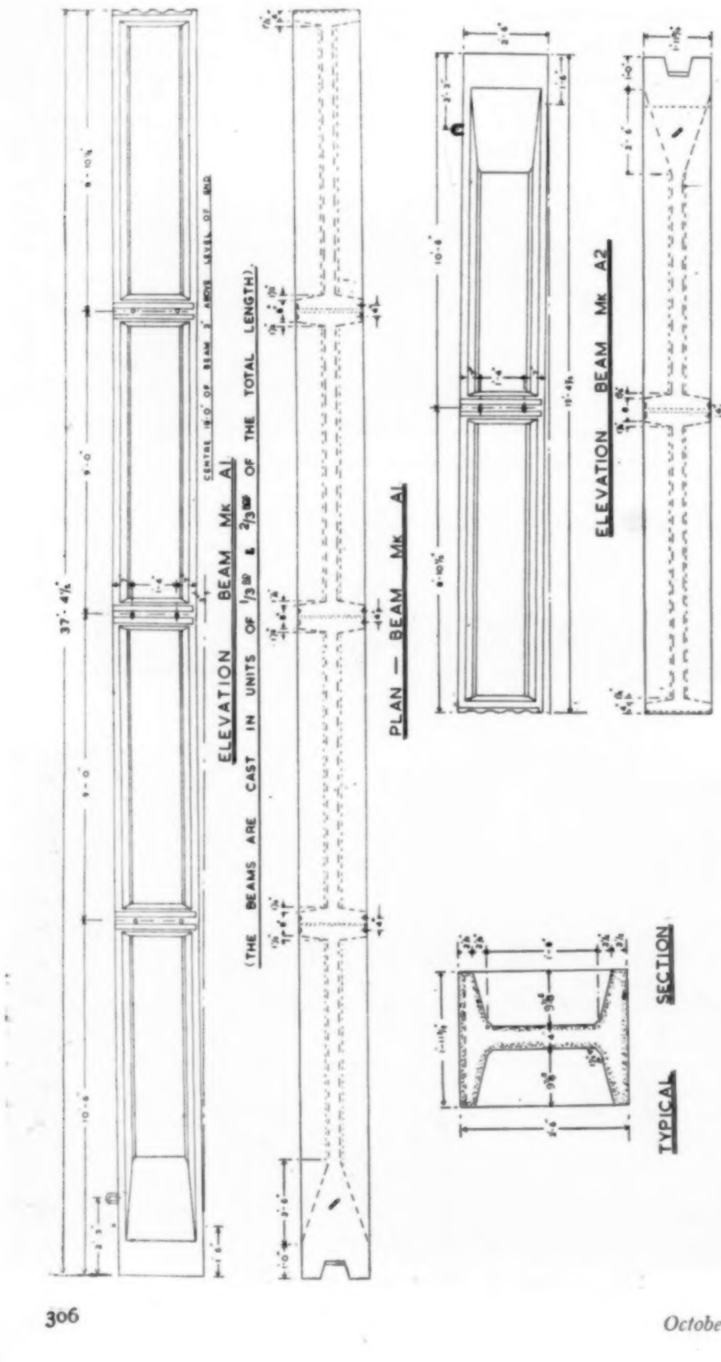


Fig. 9.—Details of Prestressed Beams.

Fig. 10.—Prestressing Beams Transversely.

the high-tensile bars are formed by inflatable rubber tubes supported in position in the shuttering by precast concrete inserts. The beams, in their moulds, are then transported by a gantry crane to the curing area and subsequently to a stacking area, where they remain until a strength sufficient to withstand the post-tensioning forces has been developed. This is determined by cubes cast at the same time as the beams and which are tested at the works. A compressive strength of at least 4500 lb. per square inch is required before tensioning the bars.

Before either of the two parts required to make one beam 57 ft. long is moved a mild steel bar, threaded at each end and with a nut and plate anchorage, is passed through the lowest cavity formed for the high-tensile steel bars. The bar is tensioned with a force of 2½ tons, which is sufficient to enable the beam to be lifted safely. When the concrete has reached the required strength the beams are transported by crane to the stressing beds, where they are laid on timber

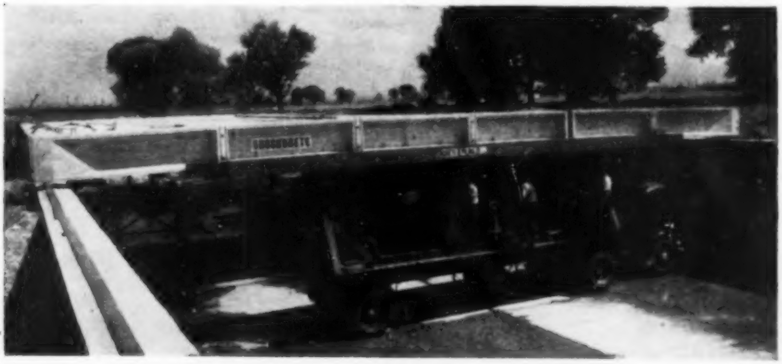


Fig. 11.—Transferring Beam from Lorry.

Fig. 12.—Beam in Position.

bearers levelled so as to form a camber of 3 in. at the centre of the beams and the temporary prestressing bars are removed.

The joint between the two parts is then made, using high-alumina cement concrete to enable the final stressing to be carried out as quickly as possible. The high-tensile steel bars are passed through the cavities formed to receive them and the anchoring devices are adjusted. The bars are then tensioned equally from each end to reduce the losses due to friction, the top bar being tensioned first, then the bottom bar, and finally the centre bar (*Fig.* 14). As the tensioning of the second bar causes a longitudinal deformation of the beam and thus reduces the force exerted by the bar first tensioned, and similarly the last bar to be tensioned reduces the force in the two bars previously tensioned, allowance is made for this by stressing

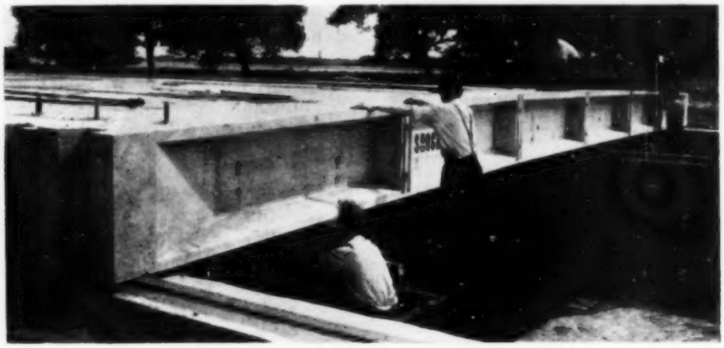
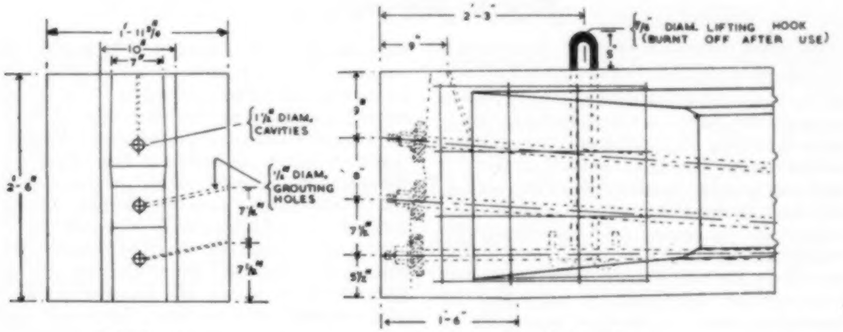


Fig. 13.—Final Adjustment of Beam.

Fig. 14.—Tensioning the Longitudinal Bars.



End Elevation.

Side Elevation.

Fig. 15.—Details of Ends of Beams.

the first bar to 45 tons per square inch, the second bar to 43½ tons per square inch, and the third bar to 42 tons per square inch. It is considered that the final force in each bar is about 42 tons per square inch. In the Lee-McCall system the ends of the bars are threaded and special nuts on these threads bear against the steel end-plates and form the anchors for the tensioned bars.

Immediately after the bars are tensioned the space between the bars and the concrete is filled with cement grout under a pressure of 50 lb. per square inch (*Fig.* 16), water under pressure being first passed through the cavities to clear them and to lubricate them for the easier passage of the grout. Finally the ends of the bars, the nuts, and the anchor-plates are coated with bitumen ; the recesses at the ends of the beams to accommodate the anchoring devices are filled with concrete on the site. The beams are then transported by crane to railway wagons on an adjacent siding from whence they are delivered to the site.

The design and supervision of the work are by the Lee Conservancy Catchment Board under the direction of Mr. M. Nixon, M.B.E., B.Sc., A.M.Inst.C.E., the Engineer of the Board ; the design for the prestressed concrete beams was prepared by Mr. Donovan Lee, M.Inst.C.E., in collaboration with the engineers of the Board. The main contractors are Concrete Piling, Ltd. The prestressed concrete beams are made and supplied to the main contractors by Shockcrete Products, Ltd., of Hoddesdon, Herts.

Fig. 16.—Grouting of the Bars.

Book Reviews.

"Prestressed Concrete Structures." By August E. Komendant. (London: McGraw-Hill Book Company. 1952. Price 42*s.* 6*d.*)

BOOKS on prestressed concrete have in general followed a somewhat similar pattern. This one is different in that the author deals not only with the bonded wire and grouted cable types of beams but also with trussed beams and girders used largely in Germany and developed by Finsterwalder and Dischinger. In discussing the elastic and plastic deformations of concrete the approach is that due to Freyssinet based upon the principles of thermodynamics. The theory of design of prestressed concrete structures is developed in detail. The notation does not make for simplicity of presentation, but the methods given deserve study. A chapter is devoted to prestressed concrete shell roofs, cylindrical tanks, and silos, and includes an example of the design of a circular reservoir with a domed roof. The analysis of primary and secondary stresses in prestressed trussed beams, both simply supported and in continuous spans, is also illustrated by examples. The book is an interesting addition to the literature of prestressed concrete.

Publications of the International Association for Bridge and Structural Engineering. Volume XI. 1951. (Zurich. The Association.)

THIS volume contains sixteen papers on developments in structural engineering and the theory of elasticity, including (in the English language): "Vierendeel Truss Analysis Using Equivalent Elastic Systems," by L. A. Beaufoy; "Corrugated Concrete Shell Structures: New Developments," by K. Billig; "Two Highway Bridges with High Grade Steel Reinforcement," by A. Holmberg, and "Deformations of Reinforced Concrete," by A. I. Johnson. In the French language (with summaries in English) are: "Bending of Rectangular Anisotropic Slabs Supported Freely at Two Sides," by H. S. Gedizli; and "Stresses at the Ends of Prismatic Bodies Loaded on their Side Surfaces," by Y. Guyon. In the German language (with summaries in English) are: "The Exact Membrane Theory of Prismatic Structures composed of Thin Plates," by E. Gruber; and "Stresses in Ceilings with Panel Heating," by E. Melan.

The paper by M. Guyon deals with the evaluation of stresses in a member of small dimensions under the action of one or more concentrated forces; it is of direct application to the design of anchorage blocks for prestressed concrete members, and its value is enhanced by the inclusion of tables and graphs which enable the values and distribution of the stresses to be obtained with little difficulty.

In "Deformations of Reinforced Concrete" Mr. Johnson gives the results of his investigations, at the Royal Institute of Technology, Stockholm, into the deformations which occur at working loads and demonstrates that these may be calculated by deducting from the values given by the ordinary theory, disregarding the concrete in tension, an amount which expresses the influence of the concrete in tension between cracks. Although the latter is difficult to determine it is possible to place upper and lower limits on it and thus obtain a range of values in which the final deformation will lie. It is further demonstrated that even a relatively large deviation from the reinforcement designed in accordance with the ordinary theory in statically indeterminate structures produces only a slight effect on the magnitude of the deflection and on the distribution of moments under the action of long-period loading if the total amount of reinforcement is kept constant and its yield point is nowhere exceeded. This paper is also published separately as a bulletin of the Swedish Cement and Concrete Institute, Stockholm.

"Reclamation in the United States." By Alfred R. Golzé. (London: McGraw-Hill Book Co. 1952. Price 68*s.*)

THE author is the Director of Programmes and Finances of the United States Bureau of Reclamation and his experience enables him to give a comprehensive treatment of the subject. The book stresses the economic rather than the engineering aspects of irrigation and hydro-electric power generation, and traces the history of reclamation work in the U.S.A. from early times to the present day.

Book Received.

"Street Lighting." By J. M. Waldram. (London: Edward Arnold & Co. 1952. Price 65*s.*)

Shell Roof Construction in Belgium.

At the Symposium on Shell Roof Construction held in London in July last, M. Carlos Wets and M. André Paduart described a self-supporting vault built by the Belgian firm S.E.T.R.A. at the port of Antwerp. This structure, shown in *Figs.* 1 and 2, is 1525 ft. long divided into 31 bays of about 49 ft. 3 in. and is 199 ft. wide composed of three bays of 58 ft. 6 in. and two cantilevers of 11 ft. 6 in.

The thickness of the vault increases from 3·15 in. at the crown to 4·75 in. at the springings. The centre-line of the slab corresponds to the funicular line for its own weight and for a uniformly-distributed snow load. Wind and non-uniformly-distributed snow cause deflections which produce very slight tensile stresses in the concrete. The reinforcement is a mesh placed in the middle of the slab. Each vault rests on columns without intermediate beams and behaves like a shell supported at isolated points. For spanning transversely from column to column, the vault must be provided with special reinforcement (*Fig.* 3). The thrust of the vaults is resisted at each end of the building by four reinforced concrete counterforts.

The flexibility of the vaults in the longitudinal direction is such that deformations due to shrinkage and temperature variations cause only very slight tensile stresses in the concrete and it is possible to omit expansion joints. The intermediate columns are designed to resist the differences in thrust between two adjacent vaults because of wind action or unequal distribution of the superimposed loads.

The vaults are designed to accommodate large openings for roof lights. These openings separate the vaults into half-shells each 131 ft. long, which support one another through reinforced concrete beams, 12 in. by 8 in., at about 15 ft. centres and having the same curvature as the vaults. The reinforcement was designed to resist eccentric compression. Along the sides of the roof lights are reinforced concrete kerb beams which stiffen the vault and prevent the entry of rain or snow under the glass. As the weight of the roof light including the stiffening beam is equal to the weight

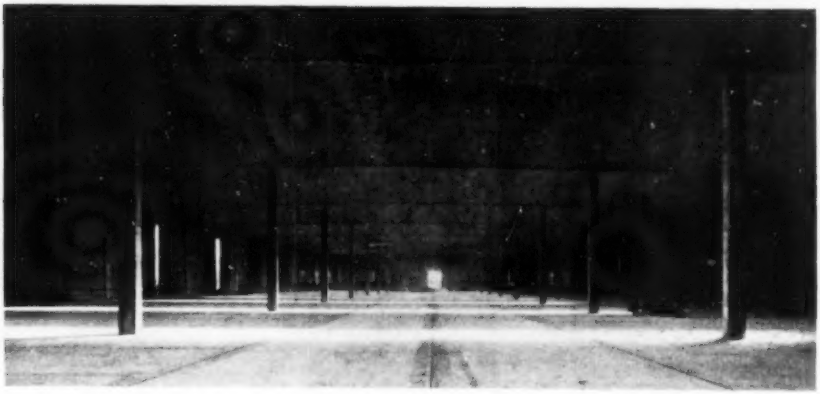


Fig. 1.—Interior of Shed.

Fig. 2.

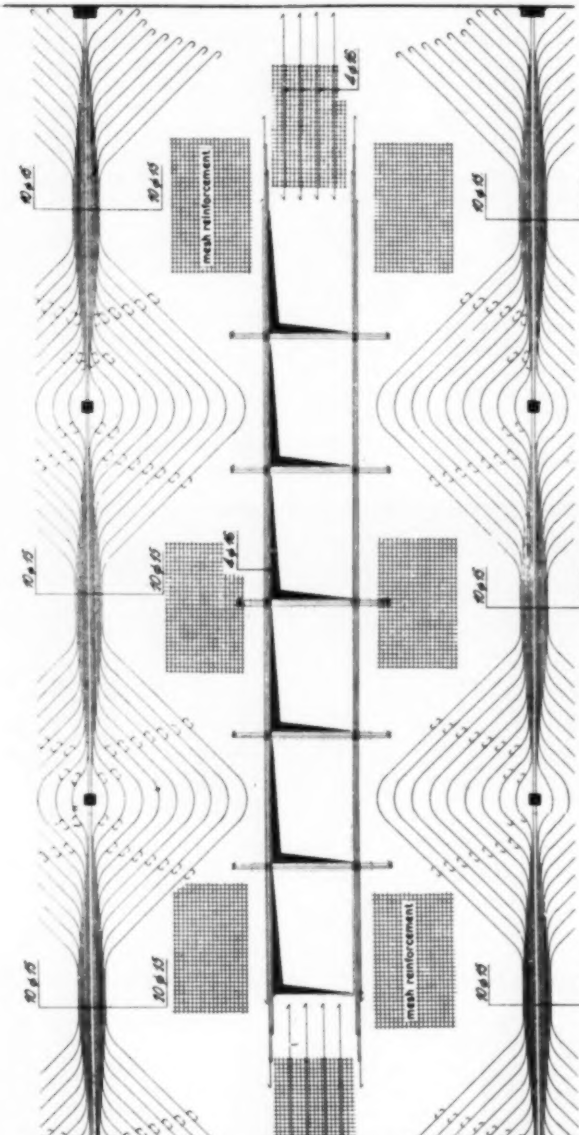


Fig. 3.—Details of Reinforcement.

Fig. 4.—Temporary Ties.

of the concrete which would have been needed had there been no opening, the funicular line was the same for the vault in the region of the roof lights as in the portions without roof lights.

For protection from wind-driven rain, the sides of the shed are closed above gutter level by thin reinforced concrete slabs supported on cantilever beams attached to the ends of the gutters. At the top these slabs fit into a groove formed in the soffit of the vaults. The roof has an asphalt finish, and transversely to the building the vaults have a joint at the centre with a slope of 1·5 per cent.

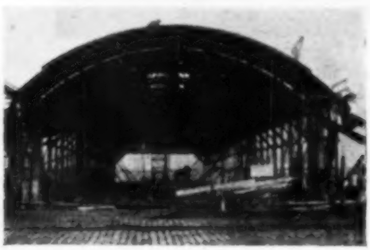


Fig. 5.—Centering in position.

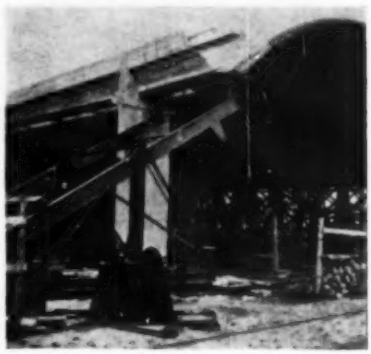


Fig. 6.—Carriage Raising Centering.

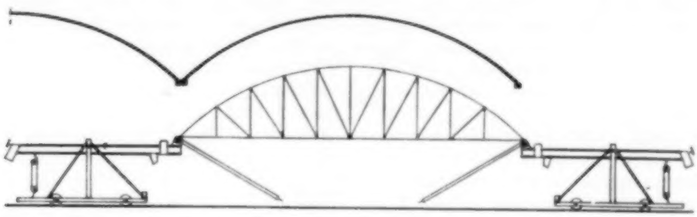


Fig. 7.—Carriages for moving Centering.

towards the edges. The downpipes are attached to the exterior columns.

The roof lights were precast and have glazing bars and reinforced concrete ridge-cappings. They are fixed by metal pins embedded in mortar.

As the 31 vaults lean against one another and their thrust is resisted by the end counterforts, and as one centre was used for all the vaults, precautions had to be taken to absorb the thrust of the vaults during the course of the work. Metal centering was built for one bay (*Fig.* 5) to enable an area of 199 ft. by 49 ft. 3 in. to be concreted. This centering was composed of 14 parts about 13 ft. wide with plates inserted between them. As the work progressed, each part was moved in the longitudinal direction of the shed and used 31 times. Before removing the centering of a bay, the heads of the external columns were tied to the feet of the adjacent columns by four $1\frac{1}{2}$-in. diameter bars stressed to 7000 lb. per square inch (*Fig.* 4). This stress was used to reduce the elongation of the tie-rods and, consequently, the movement of the column-heads during the striking of the centering.

Each time the centering was moved it was lowered about 11 ft. 6 in. to allow the highest part to pass under the valley gutter and moved 49 ft. 3 in. parallel to the axis of the building before being raised to the next position. This was done by means of two travelling carriages. Each carriage had two arms pivoted near the middle and able to swing in a vertical plane. One end of each arm was attached to the centering and the other was

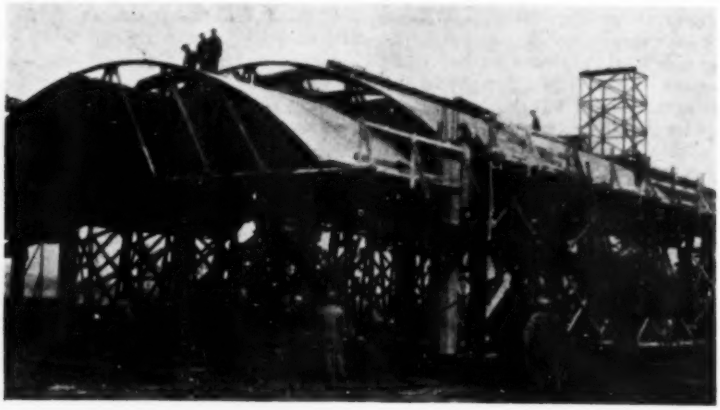


Fig. 8.—Moving the Centering.

provided with a counterweight (*Fig.* 6). The carriages were placed facing each other under each end of the centering to be moved. When the centering was attached to the arms the supporting struts were raised; the arms were then swung simultaneously thus lowering the centering which came to rest on the carriages. The centering was then moved 49 ft. 3 in., and, by swinging the arms in the opposite direction, the centering was raised to the desired position (*Figs.* 7 and 8). The centering rested on two metal struts which pivoted about their upper ends during the lowering of the different parts. This equipment enabled the centering to be moved very rapidly, and under normal conditions the centering, steel-fixing, concreting, and striking of the centering for one bay was accomplished in a week.

The longitudinal flexibility of the vaults allowed expansion joints in the roof to be omitted with the covering-strips and fillet-gutters which are often a weak point in the water-tightness of the roof covering. The curvature of the roof also increases the run-off slope and prevents water collecting. The gutters are wide and allow the water to flow away rapidly in the event of heavy rains or snow.

Two other sheds, also at Antwerp, and covering an area of approximately 29,000 sq. yd. were constructed recently in the same manner re-using the vault centering which was employed in the construction of the shed described. Each has 17 bays of 49 ft. 3 in. span, and the free height is 21 ft. 4 in. which necessitated an increase of 5 ft. in the length of the supporting struts. The construction of the three sheds covering an area of nearly 60,000 sq. yd. was carried out in little more than two years.

[The method of calculating the stresses in this structure is described by M. André Paduart on page 297 of this number.]

Lectures on Concrete.

THE following lectures have been arranged by the Ministry of Works. Admission is free.

Essentials of Good Concreting. By E. E. H. Bate, M.B.E., M.C. Croydon Polytechnic, Selhurst Road, London, S.E.25. October 13. 7.30 p.m.

Surface Finishes of Concrete. By J. G. Wilson. Gas Offices, North Street, Cheltenham. October 15. 7.30 p.m.

Concrete Placing and Formwork. By L. J. Murdock. Technical College, Cole Street, Scunthorpe. October 16. 7.15 p.m.

Prestressed Concrete. By R. W. Pearson. 22 King Street, Manchester. October 16. 7.15 p.m.

Prestressed Concrete. By F. Walley. College of Arts and Technology, The Newarke, Leicester. October 29. 7.15 p.m.

Thin Concrete Walls and Roofs.

THE types of structures described have been developed by the War Office E10 (Research).

The building shown in *Fig.* 2 is a prototype structure, 30 ft. by 20 ft. by 10 ft. high, built in the year 1951. The walls were constructed by spraying, with a cement gun, cellular concrete to a thickness of 1 in. on expanded metal used as centering and reinforcement. The cellular concrete was composed of a 3 : 1 sand and cement mixture with the addition of a foaming agent. The roof is a barrel vault of 20 ft. span, 1 in. thick, weatherproofed by a bituminous coating reinforced with fibre. The method of building the walls was as follows. On a concrete footing reinforced with ¼-in. mild steel bars expanded metal was erected and supported temporarily by tubular steel scaffolding. The mesh, which weighed 4¼ lb. per square yard, was formed on the site into corrugations 12-in. deep with a 5-ft. pitch. On each face of the expanded metal was sprayed ½ in. thickness of cellular concrete. *Fig.* 1 gives details of a similar wall, 18 ft. high, designed to resist a wind pressure of 10 lb. per square foot of elevation. The roof was constructed in the same manner except that the pitch of the corrugations was 10 ft. It is thought that walls and roofs of this form may be built in Eastern countries using woven matting of split bamboo in place of expanded metal and using bamboo in place of tubular steel scaffold. The estimated cost of the walls, excluding the footings and capping beams, is about 25*s.* per square yard and the walls require 5 lb. of steel per square yard.

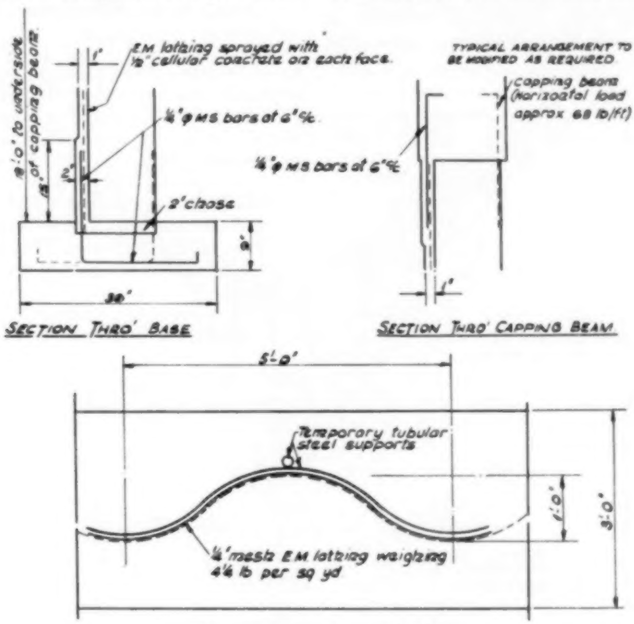


Fig. 1.—Corrugated Walls.

A trial was also made of the possibility of building cellular concrete cavity walls in a similar manner. The method adopted is shown in *Fig.* 3. First a jig is erected of tubular scaffolding or other available material and over this is secured hessian stiffened by horizontal bands of light-gauge expanded metal. The hessian is damped and sprayed with a thin coating of cellular concrete. Horizontal and vertical channel-shaped pieces of expanded metal are then fixed outside this

skin; the horizontal pieces are at the bottom and the top of the wall and the vertical stiffeners at 3-ft. 4-in. intervals. Door and window openings are trimmed with expanded metal. Wire ties are left projecting from these members to overlap the horizontal bands of reinforcement. Cellular concrete is then applied to a thickness of ¾ in. on the expanded metal channels and over the thin internal coating previously applied. Horizontal bands of light (steel mesh) reinforcement spaced at 2-ft. centres are fixed externally and hessian attached and damped. A thickness of ¾ in. of cellular concrete is sprayed externally to form the outer skin

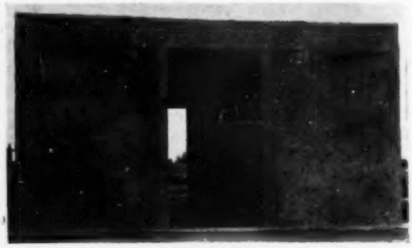


Fig. 2.

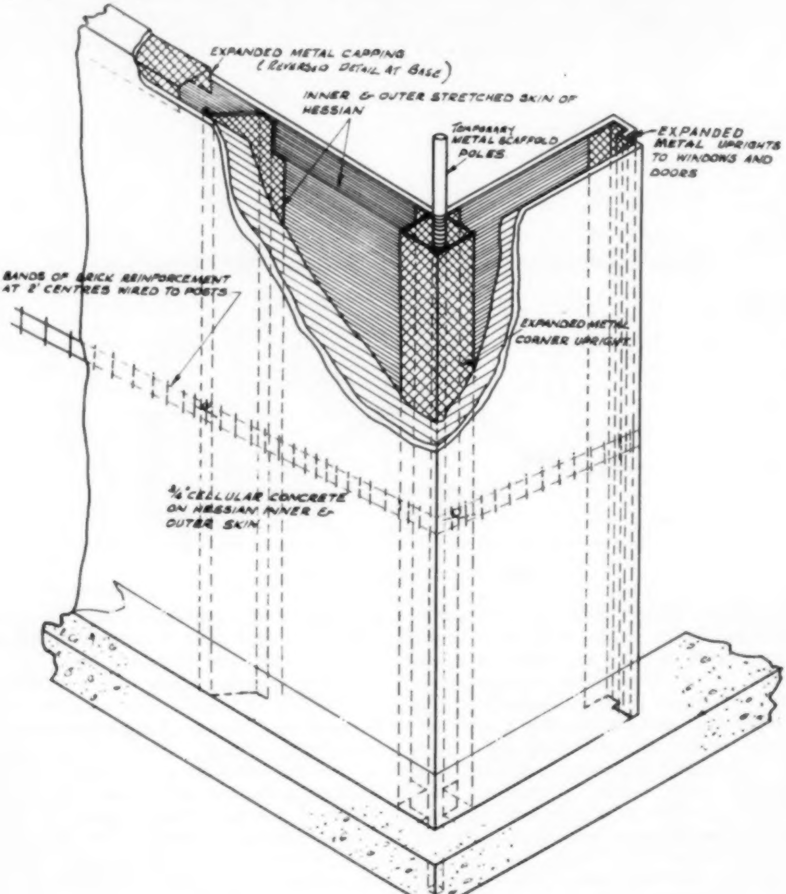


Fig. 3.

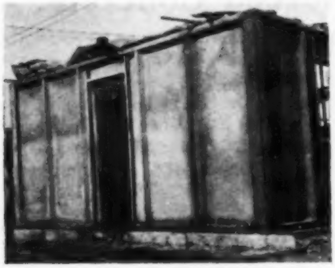


Fig. 4.—Internal Leaf of Cavity Wall.

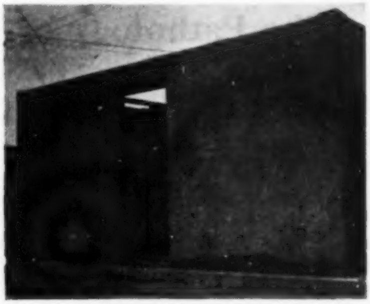


Fig. 5.—External Hessian Sheet for Cavity Walls.

of the wall and the internal jig is removed. *Fig.* 4 shows the internal skin completed and *Fig.* 5 the hessian fixed externally before the final spraying. It is claimed that this form of construction is economical because skilled labour is not required, that it combines lightness with strength, is particularly suitable for buildings on unstable ground, and has good thermal insulation properties. It is also considered that the walls could be used for two-story structures if the expanded-metal vertical members were filled with concrete.

Construction of store buildings using the " C'tesiphon " principle, as developed by Major J. H. de W. Waller under the name " Nofrango ", but using a cement-gun for applying the thin concrete cover, has also been investigated. The simple single-span form of this type of roof was described in " Concrete and Constructional Engineering " for August, 1946. Designs are being considered for multiple-bay structures of this nature using prestressed precast valley beams of 48 ft. span to carry C'tesiphon arch roofs of 25 ft. span with a rise of 6 ft. The arches have transverse corrugations with a depth of 8 in. and a pitch of 4 ft. The steel required for these roofs, including the valley beams and the columns, is less than ¾ lb. per square foot.

These notes on the work carried out by the War Office are abstracted from a paper on shell concrete by Captain J. H. Foster, R.E., of the Directorate of Fortifications and Works.

Fire Resistance of Thin Slabs.

In a recent report published by the National Bureau of Standards of the U.S.A. a description is given of fire endurance tests of slabs and partitions made of concrete sprayed on to wire-fabric reinforcement. The slabs were about 2¾ in. thick and the partitions about 2½ in. thick.

The slabs were made with aggregates composed of sand and wood sawdust, the sawdust ranging from 0 to 50 per cent. by volume of the aggregate. The aggregate for two partitions was sand, a third had a minor amount of asbestos added, and that of the fourth consisted of equal volumes of sand and sawdust. Fire-endurance limits for the slabs increased almost linearly with increased percentages of sawdust. Partitions made with sand aggregate or with the addition of a small amount of asbestos fibre showed early failure by spalling and holing. The violence of the disruptions suggested the presence of water in the dense concrete. A partition made with equal volumes of sawdust and sand as aggregates did not spall and failed by a limiting 325 deg. F. rise of temperature at a single point on the unexposed side after 70 minutes' exposure to fire, whereas the partitions having asbestos and sand or sand only as the aggregate reached fire-endurance limits through explosive spalling in 16 to 26 minutes. Subsequent to the fire-endurance test, the partition with sawdust successfully withstood the hose-stream test. The sawdust lowers the thermal conductivity of concrete and increases its porosity, thus making easier the escape of moisture in the form of vapour.

The Pathology of Reinforced Concrete.*

By HENRY LOSSIER.

CONSTRUCTIONAL DETAILS.

Combinations of Shapes.

It would seem that if two parts of a structure, each stable in itself, are combined, the whole ought to be equally stable. Such, however, is not always the case. Consider, for example, a self-supporting vault (V) and an edge-beam (R) of appreciable depth and designed to carry its own weight and that of the roof (*Fig.* 20). Because the two elements (V) and (R) are built together, when the centering was removed the vault exerted a horizontal eccentric thrust on the beam which it turned over by torsion and subsequently fractured. This is a classic example of the danger of assembling component parts having different deformations.

Shallow Foundations.

In some parts of the world, and particularly in North Africa, there is no alternative to placing foundations near the surface on compressible or unstable soils owing to a firm stratum being too deep for a normal foundation. Problems of this nature can be solved in a variety of ways.

In the case of a big apartment building in Tunisia, for example, an extensive basement was constructed with the object of excavating a weight of earth equal to that of the building and thereby causing no interference with the general equilibrium of the sub-soil. As tilting of the hollow raft could none-the-less occur during the progress of the work, bags of sand were kept in readiness and used as movable kentledge for righting it as occasion demanded. Subsidences averaging 3 ft. deep are not uncommon in certain countries.

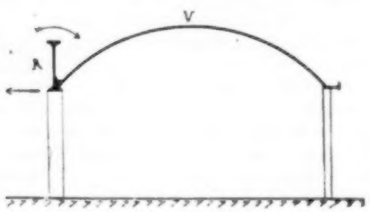


Fig. 20.

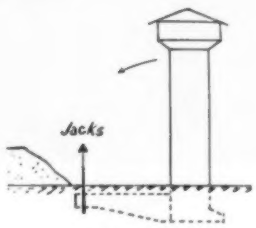


Fig. 22.

At Casablanca, where a factory was built on recently reclaimed and unstabilised ground, the following two procedures were adopted. (*a*) Half-submerged lifting-galleries were formed by means of short sections of reinforced concrete coupled together by joints capable of ensuring strength and watertightness in spite of considerable unequal movement (*Fig.* 21). (*b*) The foundations of a tower were extended to enable it to be maintained perpendicular by the use of jacks (*Fig.* 22). Subsidences of up to 3 ft. have not resulted in harmful effects.

At several sites columns have been built with their bases inserted freely in boxes of reinforced concrete integral with the foundations. Very simple devices for lifting by jacks are used as required to correct settlements. An example is the deck of a quay at Sfax, where the various parts can be lifted separately (*Fig.* 23).

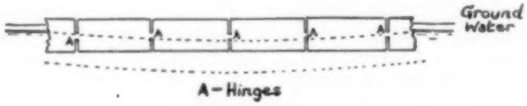


Fig. 21.

* Concluded from July and August numbers.

A particularly difficult case occurred at a port. Under a top layer of filling is a thick stratum of mud which not only undergoes considerable settlement but also moves horizontally towards a dredged river, thus necessitating adjustment in two directions; this was achieved by having enough clearance between the columns and the sides of the boxes in which they are erected. When this ground was loaded for the first time with heavy material, the layer of mud, compressed between the filling and the firm soil, flowed out to the river and broke the reinforced concrete piles of a wharf. It was therefore necessary to dredge the mud, reform the site with sand, and then reconstruct the wharf with thin piers founded on the good ground in case of a recurrence of the flow of mud from a more distant part.

When foundations are on water-bearing strata it is necessary to take special precautions if the buildings are to contain refrigerating plant. In the case of an industrial building in the environs of Paris the structure comprised a central building of greater height than the two flanking buildings. It was found that there was no settlement of the annexes but that the central block was slowly and continuously rising. The explanation was the presence of a cold chamber with a temperature of −30 deg. Centigrade which was badly insulated and which froze the water contained in the ground, thus raising the central block by expansion of the ice (*Fig.* 24). With the object of avoiding, on the thawing of the ground, a descending motion which might have caused more damage than the rise of about 3 in. which had already occurred, the central structure was stabilised in its new position by wells below the frozen ground, after which arrangements were made to procure a general thaw.

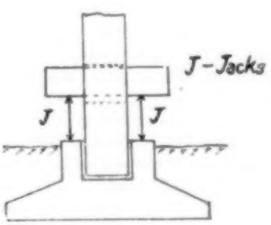


Fig. 23.

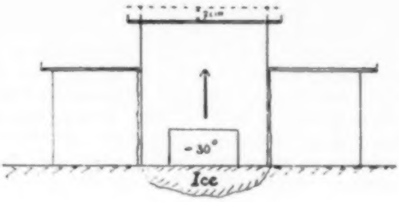


Fig. 24.

Foundations on Piles.

Piled foundations have been the cause of many mishaps, due to a great diversity of causes. If the ends of the piles are in a stratum which itself is firm but is supported by a less stable stratum, it often happens that the total load which a group of *n* piles can support is less than *n* times the strength of a single pile. It is then necessary to ensure that the spacing of the piles and the thickness of the first stratum are such that the pressure transmitted to the second stratum does not exceed the safe bearing capacity of the latter.

On the banks of the Seine, where compact chalk is usually covered by alternating layers of various materials (clay, mud, gravel, fissured or thin seams of chalk, etc.), many mistakes have been made which could have been avoided with a better knowledge of the sub-soil. The following are typical cases. At a site in Normandy piles were cast inside a metal tube, the shoe of which fitted so badly that the concrete, as it was put in the tube full of water, was washed and on arrival at the bottom of the tube had no cement in it. Under the weight of the building, the lateral pressure of the uncemented pebbles compressed the muddy clay traversed by the piles and the piles gradually collapsed. After a year the building developed a lateral tilt of 10½ in., that is a settlement of ½ in. on one side and 11 in. on the other side. The remedial measures taken were to insert a new foundation by underpinning, then to lift the building by means of jacks, and finally to stabilise the ground. This was done without disturbing the use of the building.

In several cases, precast piles have been rapidly rendered useless due either to the use of cements unstable in contact with soft water or sea water, to insufficient

cover of concrete to the reinforcement, or to lack of compactness of the concrete. In ports or tidal waters, disintegration or decomposition generally first appears in the zone of ebb and flow by reason of the mechanical action of the water which facilitates damage by acids or other industrial effluents which float on the surface owing to their low specific gravity.

Pile Formulæ.

The question of the formulæ used to evaluate the resistance of piles deserves to be examined since their interpretation has played a leading part in difficulties experienced with foundations.

Consider first the case of cast-in-situ systems in which a tube is driven or screwed into the ground and gradually withdrawn whilst the concrete is rammed. The resistance to penetration of the metal tube when it is driven by a hammer certainly gives a measure of the bearing capacity of the soil. But the finished pile does not exert the same lateral pressure as that of the tube whilst it is being driven, and nothing justifies the conclusion that the ultimate resistance of the pile is the same as that of the metal tube. The evaluation of this latter resistance is moreover itself a function of the driving formula employed, that is it is more or less arbitrary. If it is desired to pursue speculative research, one of the many methods of calculation known as "static methods", which take into account the properties of the different strata to be penetrated, should be applied.

If the piles are precast and driven with a hammer, the problem likewise presents features of great complexity. The piling formulæ, and particularly that known as the Dutch formula which has been considerably improved in recent years, give results which, when used in a rational manner, are confirmed in practice. The following examples illustrate aspects of the subject which are sometimes of considerable intricacy.

Near a French river some reinforced concrete piles 20 in. diameter and 100 ft. long were driven in poorly-consolidated ground comprising made-up material over a thick seam of muddy clay on solid rock. All the piles were driven by a drop-hammer weighing 8 tons, and the Dutch formula had given a relatively great theoretical resistance, which was interpreted as proof that all the piles had reached rock. Nevertheless after several years the piles started to sink gradually under the weight of the building. Borings made next to some of the piles of which records had been taken showed that their points had come to rest several yards above the rock. The fact that the weight of the hammer was only slightly over half that of a pile had invalidated the results of the Dutch formula, which was in fact no longer applicable; also, the incompletely consolidated ground had increased the resistance of the piles by friction and adhesion on account of its settlement, thereby further aggravating the collapse of the foundations. If the bottom bearing of the piles had been verified by borings, and if a hammer at least equal in weight to that of a pile had been used, this trouble would very likely have been avoided.

In another case precast reinforced concrete piles about 66 ft. long were driven in soft clay of very great relative depth. These piles, of necessity supported by friction only, were made in two stages, at first with an initial length of 40 ft. and then lengthened, after preliminary driving, by adding a further length of 26 ft. The contractors contended that the use of the Dutch formula implied the following anomalies. Towards the end of the initial driving, the apparent resistance diminished very greatly to the point of being only a small fraction of the stipulated working resistance. Then, when driving was resumed after several days during which the extension was made, the resistance was suddenly very much greater, and then continued to diminish until the end of the driving. If, after several days, a few blows were given for checking purposes, an apparently satisfactory resistance was encountered, but this was not confirmed when a pile was tested under a direct load.

These anomalies arose from two causes, which in this case acted together but which sometimes occur separately. In the first place the piles, which were of relatively small cross-sectional area, vibrated and even "whipped" laterally during driving, the more so as the ground offered small resistance. These movements enlarged the cavity in the ground, thus reducing the friction of their sides against the soil. Since this friction represented almost the whole of the resistance to driving, the latter was naturally less

towards the end of the driving. After a pause the soil regained contact with the pile, thereby re-establishing friction. In the second place, the soft clay stuck to the concrete during the halt in driving, and when driving was resumed added a part of its weight to that of the pile, invalidating in this manner the application of the piling formula by making the theoretical resistances too great. Under direct load the actual resistance was less than that calculated.

These cases underline two precautions to be taken when "friction" piles are driven in cohesive soil, namely, avoidance of too slender piles which may "whip" during driving, and making in every instance a direct load test to ascertain the true resistance. Account should be taken of the reduction in driving efficiency when a small-scale pile of wood or metal is used. Reductions of one-third to two-thirds are not unusual.

In the case of a reinforced concrete bridge of large span, the foundation of each pier was formed of a group of reinforced concrete piles, each 16 in. square, spaced at 32-in. centres in both directions. These piles were driven in a clay-sand soil. The carrying load P of a pile had been calculated by a variety of formulæ known as "static", and it had been assumed that the total strength of a group would be equal to nP, n being the total number of piles in each group. When the centering was removed it was noticed that the foundations were sinking under the load. The settlement having reached more than 4 in. without sign of stopping, the jacks were again used to replace the centering under the bridge, and it was necessary to carry out a very costly strengthening of the foundations. The group of piles, being too closely spaced, had acted as a single unit and the friction of the soil had acted against the outer piles only with greatly reduced efficiency (*Fig.* 25). This elementary error is by no means infrequent.

In another example, a building in a colliery district was founded on piles driven at a site where mine refuse had been dumped. An outburst of spontaneous combustion affected the strength of the steel and concrete to such an extent that serious subsidences occurred less than two years later. It was necessary, after shoring the building, to remove the refuse by stages down to good ground,

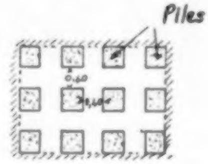


Fig. 25.

build a new foundation, replace the refuse by sand, and finally to raise and repair the structure.

In the case of a piled foundation for an industrial plant it was necessary to defer for some years the construction of the superstructure. Before resuming work, the precaution was taken of testing a pile under a direct load. The result gave a very low resistance. The foundation was on the site of a former plant for the manufacture of sulphuric acid and the impregnated soil had decomposed the steel and the concrete of the piles. It was necessary, as in the previous case, to remove the piles and the soil down to good ground, build a protection against the surrounding parts, reform a bank of sand, and reconstruct the foundations. It cannot be too firmly emphasized that enquiry should be made into the history of a site on which it is proposed to erect a structure.

The final example relates to several reinforced concrete piles for an apartment building which had resisted driving at much shallower depths than nearby piles and had settled several years later. It was found that these piles were broken in the ground during driving, their initial resistance having been brought about by the lower part acting as a distributing shoe. Their delayed settlement was caused by corrosion and fracture of the reinforcement at the break.

Floor Beams.

Strengthening expedients depend on whether the weakness is due to the horizontal bars, the stirrups, the concrete, or a combination of these defects.

In the case of longitudinal bars the method of cutting away concrete to permit of new bars being placed by the side of the existing ones, followed by reconcreting, is not recommended unless the beam is supported by the aid of jacks while the new concrete is hardening, and strongly expansive cements are used in

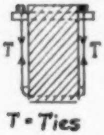


Fig. 26.

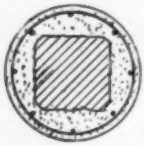


Fig. 27.

order to reconstitute an approximately normal initial state. It is possible, also, to introduce stresses contrary in sense to that of the normal stresses by prestressing.

In the case of weak stirrups alone, at a factory in the East the beams were fitted with frames put in compression by means of tightening bolts (T) as shown in *Fig.* 26. The test to destruction of a beam so strengthened has fully justified this expedient.

If the concrete alone is at fault, it is possible to prestress the beam either by compressing the stretched face or by stretching the opposite face, or finally by external binding.

Lastly, if the weakness is due to several of the causes enumerated, recourse may be had to a device comprising oblique bars which create an artificial bending opposed in sense to that of the normal bending of the structure as explained later under " Bridges ".

Columns.

The strengthening of columns, frequently necessitated by the unexpected tilting of buildings, is nearly always carried out either by the addition of steel binding placed in tension and grouted, or by a casing of reinforced concrete which has the advantage of adding its own strength whilst increasing that of the column (*Fig.* 27).

Circular Tanks.

Vertical cracks in tanks are caused by excessive stresses in the reinforcement, which shrinkage can often make worse. The cracks may either be grouted from the outside while the reservoir is full so as to open them, prestressed hoops may be added externally while the tank is empty, or an internal plastic rendering may be applied. Certain special grouts which give excellent results on tanks with thick and rigid walls are not so suitable for flexible tanks because of the fragility of these grouts.

Horizontal cracks can be caused either by shrinkage, by incomplete filling of a tank, or by the action of the base in resisting the deformation which the walls would undergo if they were unrestrained. Prestressing in the vertical direction is possible but sometimes difficult to execute. After grouting the cracks it is usual to protect the tank from variations in the ambient temperature by an earth bank, and it is especially necessary to protect it from the rays of the sun which tend to make it oval in shape and to cause secondary stresses near the base.

Bridges.

Heavy arches can be strengthened by building new arches under them and put into compression by means of jacks or expanding voussoirs (*Fig.* 28). Cellular arches are strengthened in the same manner, the new parts being placed against the interior of the cells and invisible from the outside. Some arches have been strengthened by inserting, at the crown and the springings, compensating voussoirs which were put under load by jacks or by the use of expanding cement. Bow-string bridges comprising

Fig. 28.

arches, ties, and suspenders are treated in the same way as trusses in arched roofs.

Hollow-beam bridges of constant or variable depth, with one or several spans, can usually be strengthened by prestressing. The method consists of placing, inside the cells, cables which are tensioned by jacks so that the stresses they produce cancel wholly or partially those due to bending moments and shearing forces caused by the permanent load. Hinged struts of reinforced concrete or steel eliminate all frictional stresses due to the tension of the cables. These devices have the advantage that they can be inspected and adjusted at all times, which is of particular value in providing for the increases in live load which are liable to be brought about by the increasing weights of vehicles.

Precast Concrete Revetments for Water Channels and Sea Walls.

SOME types of revetments used for lining canals and dikes in Holland are described in the following.

Blocks of the shape shown in *Figs.* 1 and 2 are hexagonal on plan with sides of 5 in. or 6 in., and are made in depths from 4 in. to 12 in. Sometimes the tops are rugged as shown to the right of *Fig.* 1, and there are bevelled edges about $\frac{3}{8}$ in. deep at the top. The smaller blocks are made on drop-hammer or hydraulic presses, while the larger sizes are cast separately in steel moulds. The blocks with rugged tops are also sometimes used for lining sea-walls.

A well-tried method of lining dikes is shown in *Fig.* 3. This comprises slabs 16 in. square by 4 in. thick with stepped edges on all four sides as shown at (*a*). They are generally supported at the bottom by a concrete kerb 8 in. wide by 12 in. high and at the top by a beam 5 in. wide by 8 in. deep; 4 in. of the depth of this beam is sunk in the ground. This is shown at (*b*), which also shows an alternative method of jointing with tongues and grooves. This is known as the De Boer system.

Fig. 1.

Fig. 2.

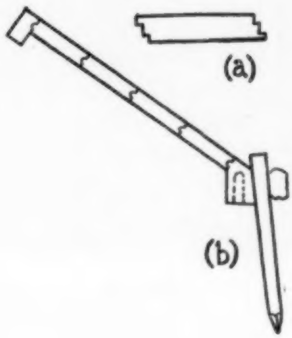


Fig. 3.

Greater resistance to damage by gales is provided by the Leendertse system shown in *Figs.* 4 and 5. Blocks are laid in the form of steps, and have been much used to protect the banks of the reclaimed areas of the Zuyder Zee. The upper face of the blocks is generally from 16 in. to 20 in. square, and they are laid to break joint.

The Oord system is shown in Figs. 6*a*, 6*b*, and 6*c*. The shape shown at (*a*) is for light work, while those at (*b*) and (*c*) are for use where there is greater risk of damage by the sea. The blocks shown in *Fig.* 6*a* are 1 ft. 6 in. long in the direction transversely to the direction of the revetment and 6 in. thick at the middle; those at (*c*) are about 12 in. thick at the

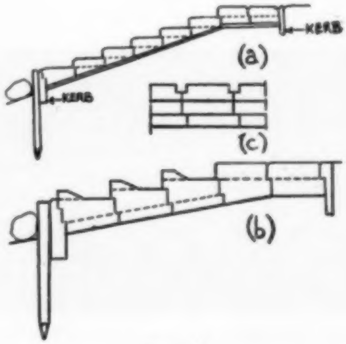


Fig. 4.

Fig. 5.

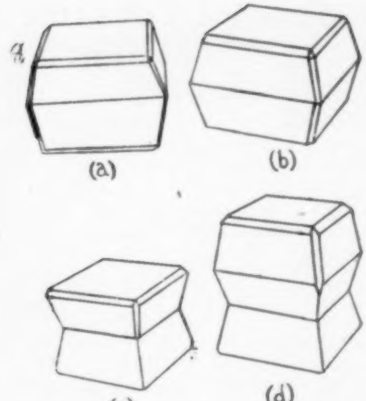


Fig. 8.

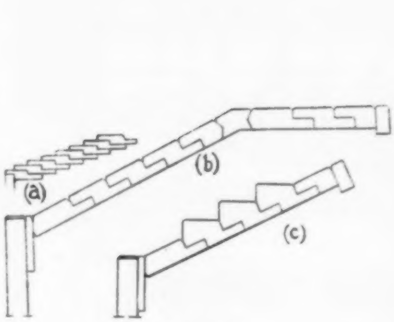


Fig. 6.

Fig. 9.

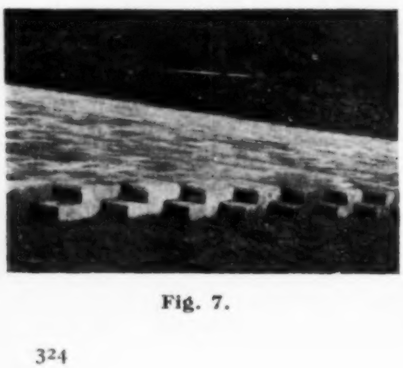


Fig. 7.

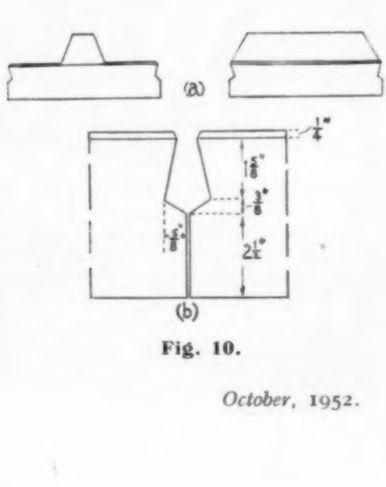


Fig. 10.

deepest cross-section. These blocks are shown in use in *Fig.* 7.

Because most types of precast revetments are liable to arching, the Streefkerk system was developed. This is shown in *Figs.* 8 and 9. Four types of blocks are used, type (*d*) projecting above the others to break the force of the waves and also, by their interlocking action with the shallower blocks, resisting the tendency to arching or settlement on unstable soil. This type has been found to be particularly useful on curved work.

Precast blocks of the shape shown at (*a*) in *Fig.* 10 were used to repair the sea-wall near Walcheren which was destroyed during the war. This is a square block with sides of 1 ft. 10 in. and 4¾ in. thick, with a rib projecting 4 in. on the top face to break the action of the waves. The joint, which is filled with hot bitumen, is shown at (*b*). The blocks are laid with the joints and ribs staggered in each course.

None of these blocks is reinforced. It is commonly specified that the absorption of water must be less than 9 per cent. of the weight of the block, and that the compressive strength must be at least 5000 lb. per square inch and on important work not less than 6500 lb. per square inch on 3-in. cubes.

Delivery of Loose Cement.

Fig. 1.

Fig. 2.

Two of the more recent types of vehicles for delivering loose cement are shown in *Figs.* 1 and 2. That shown in *Fig.* 1 is a high-angle tipper capable of carrying 15 tons of cement, and is suitable for delivering large quantities of cement which can be discharged directly to storage pits or weigh-batching plants. For smaller works and sites in built-up areas delivery by the lorry shown in *Fig.* 2, from which the cement, assisted by air, is discharged through three spouts into enclosed hoppers, is more suitable; this vehicle has a capacity of 8 tons. The Cement Marketing Co., Ltd., now has 127 vehicles in use for the delivery of loose cement from nearly all their works, and their deliveries by this method are nearly 16,000 tons a week.

Exhibition of Vibrating Equipment.

The Cement and Concrete Association has organized an exhibition of vibrating equipment of various types at their research station at Wexham Springs, near Slough. Anyone wishing to visit the exhibition should write to the Association at 52 Grosvenor Gardens, London, S.W.1.

Test of a Reinforced and Prestressed Concrete Frame.

THE frame shown in *Fig.* 1, consisting of a prestressed concrete beam and reinforced concrete columns, was recently tested at the works in Manchester of the Trussed Concrete Steel Co., Ltd. The beam (*Fig.* 2) was a prototype of those to be used in the construction of the new Medical School for Liverpool University. The building is a five-story structure about 37 ft. wide and about 190 ft. long with a wing about 65 ft. long by 37 ft. wide projecting from the main structure. The columns are at 10-ft. intervals along the external walls. The floor is to be of precast beams laid on the upper flanges of the main beams, with in-situ ties at the ends of the span. A beam of this shape was used at the request of the architects to give rooms with sloping ceilings. Large holes were made in the deepest portion of the beam to accommodate services.

The test structure was built to simulate as far as possible the conditions in the

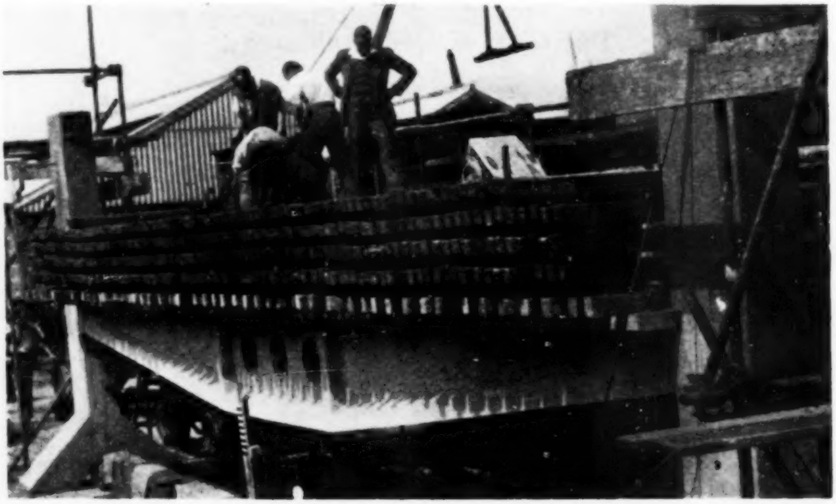


Fig. 1.

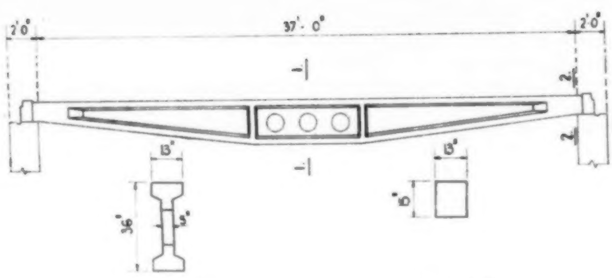


Fig. 2.

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Advertisements must reach this office by the 23rd of the month preceding publication.

SITUATIONS VACANT.

SITUATION VACANT. Chief designer required by consulting structural engineers, Westminster. Considerable experience of reinforced concrete buildings essential, and experience of structural steelwork desirable. Applications in confidence, stating age and qualifications, with full details of experience, to Box 2590, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Consulting structural engineer, Westminster, requires senior designer-draughtsman, with first-class experience in reinforced concrete, for responsible position. Experience in structural steelwork an advantage. High salary and good prospects for suitable applicant. Write in confidence, stating age, qualifications, and full details of experience. Box 2591, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Technical sales representative required to promote sales of precast and prestressed concrete units. Must be able to discuss design features with clients. Qualified engineers considered. Opportunity for early promotion to sales manager. Liberal salary and commission basis. Apply, with experience and references, to the SECRETARY, DOW-MAC (PRODUCTS), LTD., Tallington, Stamford, Lincs.

SITUATION VACANT. Engineer with 10 years' practical experience in civil engineering required for control and supervision of contracts in Singapore by J. C. GAMMON (ENGLAND), Ltd., Burwood House, Caxton Street, London, S.W.1. Pay: £2,000 per annum, with good leave pay, passage allowance, and bonus for good work.

SITUATION VACANT. Structural engineering design draughtsman required in busy design office with varied work. Concrete, steel, and general construction. Able to work with minimum supervision. Minimum age 25 years. Salary dependent upon experience. Progressive post. Write, with full particulars, to Box 2599, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Engineer required for estimating and preliminary design by large civil engineering firm in London. Preferably age 25–30. J. L. KIER & CO., LTD., 7 Lygon Place, London, S.W.1.

SITUATION VACANT. Reinforced concrete detailer-draughtsman required in London office. Write, with details of age, experience, and salary required, to STENT PRECAST CONCRETE, LTD., 1 Victoria Street, London, S.W.1.

SITUATION VACANT. Nairobi (Kenya). Experienced reinforced concrete designer required for the design of commercial structures. Applicant must be qualified structural engineer and capable of holding responsible position. 4½ years' contract, with paid passages, home leave, and pension scheme. Write Box SE/93, c/o 95 Bishopsgate, London, E.C.2.

SITUATION VACANT. Designer-draughtsman required in London head office of old-established reinforced concrete engineers specialising in hollow-type floor construction. Progressive post. Write full qualifications and salary required. Box 162, ALLARDYCE PALMER, LTD., 109 Kingsway, London, W.C.2.

SITUATIONS VACANT. Reinforcement manufacturers have vacancies in their Bristol office for reinforced concrete designers. Posts are available for applicants with good experience, and also for suitably qualified persons for training. Work involves a variety of structures. Please give fullest details when replying. Box 2601, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

DRAUGHTSMEN

Simon-Carves Limited announce the opening of an additional new drawing office in Mayfair. The firm specialise in the design and erection of Power Stations, Coal Preparation and Chemical Plant, and the new office will be devoted to reinforced concrete work. There are a large number of vacancies for designer-detailers with sound experience of reinforced concrete, and for a smaller number of men with experience of structural steelwork. Salaries are above average, a pension scheme is in operation, working conditions and scope are excellent, and free meal vouchers are provided. Brief relevant details should be sent to

Staff & Training Divn. (Ref. L11),
Simon-Carves Limited,
Cheadle Heath,
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All candidates will be interviewed in London.

SITUATIONS VACANT. Consulting engineer requires two reinforced concrete detailer-draughtsmen for Kensington office. Salary according to age and experience. Write giving particulars to Box 2600, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Reinforced concrete draughtsman wanted in consultant's office. Experience in structural steel work an advantage but not essential. Reply to MR. J. BAK, 24 Baker Street, London, W.1. Telephone: Welbeck 7091 and 2932.

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BIG BUILDING PROJECTS. Advertiser wishes to contact manufacturers of building construction machinery, moulds, steel shuttering, flush sewage systems and chemicals, and drainage and water-supply materials for constructing single and 2-story houses. TAN SWEE WAH, 180 Hugh Low Street, Ipoh, Perak, Malaya.

EXCHANGE.

REQUIRED. ⅜-in., $\frac{5}{16}$-in., and ¼-in. diameters black annealed round mild steel reinforcing rods in exchange for ½-in. diameter similar. Apply RIBAR, LTD., Higher Swan Lane, Bolton.

FOR SALE.

FOR SALE. Tubular ducting. 20,000 ft. 6½-in. O/D × 14G × 18 ft. long, at 2s. per ft., ex works. E. STEPHENS & SON, LTD., 58/64 Bath Street, London, E.C.1.

STEEL SHEETS: 30 tons 53 in. × 19 in. × 14G, painted or light rusty, at £30 per ton. GALVANISED SHEETS: 5 tons 70 in. × 33 in. × 26G, at £30 per ton. STEEL BOXES: 1,000 21 in. × 9½ in. × 9 in. deep, at £10 per 100. E. STEPHENS & SON, LTD., 58/64 Bath Street, London, E.C.1.

FOR SALE. Sacks, bags, and curing cloths for sale. You want the best type and quickest delivery. Write JOHN BRAYDON, LTD., 26 The Highway, London, E.1. Telephone: ROYal 1044.

FOR SALE. Reinforcing rods. Light diameter materials on reasonable delivery against ISA, subject unsold, at very keen prices ex works Leeds. Also ⅜-in. diameter in rotation. A. BIRCHALL, LTD., 23 Farrar Lane, Leeds, 6.

FOR SALE. Four 2 cu. yd. Muirhill dumpers, £125 each. One 25 cwt. three-way Austin van, £600. One 10/12 cwt. Bedford goods van, £400. One Ford Thames lorry, model 7V, £100. Apply Box 2602, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

actual building, and materials were obtained from the same source as those for the building. The beam contained two cables each of twelve wires of 0·276 in. diameter. It was supported at each end on reinforced concrete columns the reinforcement in which extended upwards into short columns tied back to the foundations. The beam was cast in situ on the lower portions of the columns, stressed at the age of five days, and subsequently grouted. The upper columns were then cast, the reinforcement and concrete surrounding the ends of the beam providing continuity between beam and columns. The columns were designed to provide a restraint during the test similar to that provided by the building frame. The bending moments transmitted to the columns were obtained from the force in the tie-rods between the upper columns and the foundation. As the beam was loaded the tops of the columns tended to move inwards and the pull required to bring them back to their original position indicated the value of the shear force on the column and thus the restraining moment. The vertical deflections of the beam were measured.

The floors are designed for 60 lb. per square foot dead load and 80 lb. per square foot superimposed load, giving a total uniformly-distributed load of 23·6 tons per beam. The beam was loaded with copper ingots at the age of fourteen days. The first crack appeared when the load equalled the designed dead load plus twice the superimposed load, and the beam failed at a load of 60 tons which represents the final dead load of the beam when erected (including floors and finishes) plus three and three-quarter times the superimposed load.

The concrete mixture was about 1 : 1½ : 3 by weight and the strength of cubes at seven days was 7400 lb. per square inch. The working stress in compression was assumed to be 2000 lb. per square inch.

The architects for the building are Messrs. Weightman and Bullen, F./F.R.I.B.A., and the structural design and construction are by the Trussed Concrete Steel Co., Ltd.

New Plant for the Contractor.

Fig. 1.

THE machine shown handling gravel in *Fig.* 1 is available with bucket capacities of ¾ cu. yd., 1 cu. yd., and 1¼ cu. yd. and with several ancillary attachments which are interchangeable. These include a dozer blade, crane attachment, fork-lift, skeleton rock-bucket, digging teeth, and snow plough, and there is also available an "all weather" driver's cab which is easily removable. Operation of the loader is by a single lever situated to the left of the driver. The driving controls are of the conventional automobile type. The engine is the new Fordson Major, which is available for use with diesel, petrol, or paraffin fuels; electric starting is fitted and electric lighting is optional. A feature of the machine (called the Hydraloader) is that the driver can tilt the bucket as it is lifted from the stock-pile and so ensure little loss of lifted material; this enables the standard bucket to be used for carrying wet concrete.

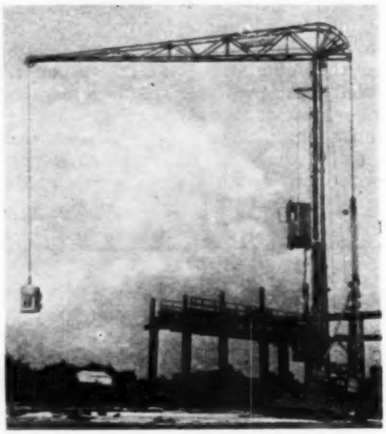

Fig. 2

The electric tower crane shown in *Fig.* 2 is of tubular construction, the mast being in three sections flanged and bolted together. The diameter at the base is 35 in. and at the mast-head casting 21 in. The jib is a welded tubular structure. Aluminium panels framed with steel angles are used for the cabin construction. All motions of the crane are electrically operated and controlled from the cabin. Four electric motors, protected from the weather, are fitted. The electricity supply required is 40 to 50 kw. at 400 volts. The travelling, slewing, hoisting, and derricking movements have separate controls which may be operated singly or in combination. All operations are stopped immediately by the electrical interlock when a limit-switch fitted to the hoisting and luffing motions is tripped. A safe-load indicator gives visible and audible warning of an overload at any position of the jib. The crane (called the Brayda) travels on rail tracks and has a maximum speed of 85 ft. per minute. Rail clamps are provided for use in stormy conditions. The cabin is raised or lowered in steps of 6 ft. by a hand-operated winch.

The capacity of the crane, fitted with a straight jib, varies from 3 tons 9 cwt. on a radius of 20 ft. and with a maximum crane-hook height of 128 ft. to 1 ton 9·5 cwt. on a radius of 65 ft. 6 in. with a maximum hook height of 75 ft. 6 in. The hook may be lowered 30 ft. below the level of the rails when using the standard cable. Working from the perimeter of a site the crane can place materials, at maximum radius, in the middle of a structure 115 ft. wide. If working from a central track the width of structure may be 131 ft. At the highest position of the cable there is adequate vision to load over an obstruction 50 ft. in height. It is stated that the crane may be erected by eight men in sixteen hours and dismantled by the same number of men in twelve hours.

The equipment described is supplied by Messrs. W. E. Bray & Co., Ltd., of Feltham, Middlesex.

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